STUDY OF PIPING FAILURES AND EROSION DAMAGE FROM RAIN IN CLAY DAMS IN OKLAHOMA AND MISSISSIPPI

A report prepared for the
United States Department of Agriculture
SOIL CONSERVATION SERVICE

by

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Consulting Engineer
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Authorization

The studies summarized in this report were carried out under a contract between the Soil Conservation Service and James L. Sherard, Consulting Engineer, dated December 18, 1970.

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PURPOSE OF STUDY

During the last 20 years the Soil Conservation Service has constructed approximately 1500 earthen flood control dams in the state of Oklahoma. Of these 1500 dams, 11 have failed by progressive erosion of a leak which developed through or under the dam. In most cases, no strong evidence pointing to a single failure cause was found and, while various conclusions and opinions were formed, it was not possible to be completely confident that the main cause of the failure was known. In general, the post-failure studies revealed no conditions which were greatly different from those of the average of many other similar dams constructed in the area.

It was well known that most of the failed dams were constructed in a general area of Oklahoma in which the surface clayey soils are especially susceptible to erosion by a process which has been characterized "dispersive" piping. In almost all of the post-failure studies it was concluded that the existence of this peculiar dispersive clay in the dam embankment was at least a contributary cause.

In addition to the 11 failures a number of the clay dams in the same general area have been damaged by a peculiar type of surface erosion from rainfall, in which vertical tunnels develop, which trouble is also caused by the dispersive tendency of the clayey soil.

A similar, and even more severe, problem of damage by vertical tunnel erosion from rainfall has been experienced in a considerable number of SCS dams constructed in a relatively limited geographic area in north-central Mississippi. Also in this area three dams have failed by breaching when the reservoir filled for the first time. A similar problem with rainfall erosion has developed in a few dams in a localized geographic area in northeastern Arkansas.

The main purpose of this study was to make an independent review of the above experiences and to form opinions concerning the probable causes of the failures and erosion damages. It was hoped that a restudy of the whole group of piping failures would reveal patterns that were not apparent during studies of the individual failures.

During the last 10 years some researchers, particularly in Australia, have shown that the erodibility of clayey soil is strongly dependent on the clay chemistry, and cannot be measured by the soil mechanics tests commonly employed under present practice by the civil engineer in designing earth dams. A summary of the recent published researches relating clay chemistry and susceptibility to erosion is given in Appendix A. Another purpose of this investigation was to examine the SCS experience in light of the results of the Australian research.

WORK CARRIED OUT

The studies summarized in this report were carried out between January and July, 1971.

In the course of the work, the writer made a number of trips to inspect and take soil samples from SCS dams in Oklahoma, Arkansas and Mississippi and trips to take samples of dams owned by others in Illinois and California. In all locations the writer received great assistance and cooperation from the Soil Conservation Service staff (see Acknowledgements).

In Oklahoma, visits were made to all the dams which had failed, to some which were damaged by rainfall erosion and to a few others. At the state office in Stillwater, the writer studied records of the design and construction of the failed dams and reports of the post-failure investigations. Discussions were held with SCS design and construction engineers concerning the evolution of practice and current details. In Oklahoma also a visit was made to the Tulsa District of the Corps of Engineers and to the site of the Wister Dam which has had problems similar to those of the SCS dams.

In Mississippi and Arkansas visits were made to inspect and take soil samples from a considerable number of dams which have been damaged by vertical tunnel erosion from rainfall.

The soil samples taken from dams by the writer during this investigation were generally obtained with a shovel or hand auger from shallow excavations on the dam slopes or old borrow areas, at locations as described in Appendix C.

In addition to those from the SCS dams, soil samples were obtained for testing from a number of other dams falling generally in two main categories: (1) dams which have been damaged by piping; and (2) dams which have not piped. The latter were included in the study to provide a "control" for the purpose of allowing a comparison of the laboratory test results of dams of satisfactory and unsatisfactory performance. A total of 102 soil samples were obtained and tested from 34 dams including

the SCS dams and others. Descriptions of these dams are given in Appendix B.

The samples were tested both in the SCS Soil Mechanics Laboratory in Lincoln, Nebraska, for the standard engineering identification tests and others (Appendix C) and in the SCS Soil Survey Laboratory in Lincoln, Nebraska for cation exchange capacity and soil chemical properties (Appendix D).

The writer also took samples of the water from the reservoirs of the Oklahoma dams which had failed by breaching and these were tested in the Lincoln Soil Survey Laboratory for total salt content and dissolved cations (Appendix D).

OKLAHOMA BREACHING FAILURES

Exhibit 1 summarizes the main features and events relating to the 11 SCS Oklahoma dams which failed. It also includes details of the Army Engineers Wister Dam and of the SCS Leader Middle Clear Boggy Site 29 Dam, which probably was on the verge of failure in 1970, and is being repaired in 1971.

Main elements of the SCS experiences are summarized under the following subheadings.

Time of Failure

Ten of the eleven dams failed on the first reservoir filling, usually within a short time after completion of construction. In general, the reservoirs filled very rapidly (within several days or less) following a rainstorm and the initial concentrated leak appeared within a few hours (or a day or two).

The first two of the eleven failures (Owl Creek Sites 7 and 13) occurred May 17, 1957 during the same storm. The other failures occurred at periodic intervals thereafter without any discernible pattern, the most recent being November 1970 (Upper Clear Boggy Site 50).

One of the 11 dams (Leader Middle Clear Boggy Site 15) failed on two separate occasions, in 1965 and 1968. The failures were at different locations and apparently were not related.

Eye-Witness Observations

At only three of the failures were eye-witnesses present to observe the events. The failures of the other 8 dams were discovered only after the reservoir had been emptied through the failure breach, and sometimes the time of the failure was not known within several days.

Locations of Failures

The failures started with an initial leak which gradually eroded a tunnel. When the tunnel became large enough the roof fell in forming a breach.

Appendix F contains longitudinal sections showing the locations of the failure tunnels and breaches with respect to the subsoil profile. From these sketches the following conclusions are drawn:

1. With the single exception of Cherokee Sandy Creek Site 8-A (Sketch F-4) all the failures occurred where the depth to bedrock in the foundation was not great and was changing relatively rapidly. Hence, with the single exception, failures occurred at a point where relatively high differential foundation settlement would be expected.

2. In two cases, Caney Coon Creek Site 2 and Leader Middle Clear Boggy Creek Site 33, two independent tunnels occurred more or less simultaneously and apparently independently (see Photos 17 and 18, Appendix J).

Including these dual tunnels, the two failures at Leader Middle Clear Boggy Creek Site 15 and the potential failure at Leader Middle Clear Boggy Creek Site 29, there are a total of 15 piping tunnels or breachs in the 12 dams. Of these 15, seven were located at or very near the location of the conduit passing through the base of the dam.

3. In the case of Little Wewoka Creek Site 17, where an eye-witness was present to observe the events from the initial development of the leak, it was clear that the leak started through the embankment at about mid-height and then gradually eroded downward to expose the conduit, so that it is well known that the initial leak did not develop along the

conduit even though the conduit was exposed finally. A similar situation apparently nearly occurred at Leader Middle Clear Boggy Site 29 (Sketch F-12) where a muddy leak developed through the embankment at about mid-height, but directly over the conduit: if this leak had progressed to failure, very likely the erosion tunnel would have finally exposed the conduit. In some of the other cases where the conduit was exposed by the failure, it seems probable that the initial leak travelled along or near the conduit.

4. In four of the dams where the conduit was exposed by the failure the conduit was underlain by several feet of compacted earth fill placed in a trench excavated under the conduit, which trench was bottomed at or near the bedrock surface, creating a condition of potential differential settlement through the dam in the vicinity of the conduit (Sketches F-1, F-5, F-6 and F-8).

5. While some of the leaks leading to failure, not located adjacent to conduits, may have developed at the contact between the dam and the foundation, it is clear that in other cases the foundation leakage was not involved at all; for example, at the Upper Clear Boggy Site 50 the bottom of the breach is still in the compacted embankment and well above the foundation (Sketch F-11) and Photos 13 and 14, Appendix J).

6. While it is possible that in some cases the initial leak traveled in the natural foundation soil just below the dam for a portion of its length, it is clear that in no case was the failure caused by a leak which passed deep into the foundation, bypassing the dam.

Geology and Embankment Soil Types

As shown in Exhibit 2, nine of the 11 failed SCS dams are located in a relatively small area (within a circle with a radius of less than 40 miles) to the south and east of Oklahoma City. The geologic and soil conditions at all the failed dams sites are very similar. The underlying bedrock is a more or less horizontally bedded shale-sandstone formation, of Pennsylvanian and Lower Permian age, which is usually within a few feet of the surface on the dam abutments. The soils from

which the dams are built are generally clays or sandy clays of low to medium plasticity derived from the bedrock.

As seen in Exhibit 1 and in Appendix C, essentially all of the soils from which the failed dams were constructed have the Atterberg Limits and grain size distribution of ordinary clays of low to medium plasticity (CL). The only exception is Cherokee Sandy Creek Site 8A for which the typical embankment material comprises a clay of high plasticity (CH). A few of the dams contain a relatively small percentage of soils with plasticity low enough to be classified as clayey silts (ML); however, it is apparent that the average failure has taken place in embankments comprised of clay of medium plasticity (CL). Exhibit 4 shows a plot of the Atterberg Limits for all the samples taken by the writer during this investigation from the Oklahoma failed dams: it is noteworthy that not a single one of these samples classifies as ML or SM soils.

Reservoir Volumes Released During Failure and Damages

A characteristic common to all these failures was the fact that the depth of water in the reservoir and the hydraulic gradient were extremely low. The depth of water ranged from a minimum of about 5 feet to a maximum of about 20 feet with the average being in the range between 10 and 15 feet.

The volume of the reservoir at the time of failure ranged roughly between 30 and 1100 acre feet (Exhibit 1, Column 14). In not one single case was there any appreciable property damage or was loss of life even threatened. In the general case several hundred acre-feet of water were discharged more or less gradually over a period of time generally ranging between a half day to several days, so that the maximum flow downstream during the failure was of the order of several hundred cubic feet per second, which is generally not greater than the capacity of the creek bed.

Because of the small size of the average reservoir it is probable that only in the case of Caney Coon Creek Site 2 was there any real risk of a damaging flood downstream. At Caney Coon Creek Site 2, which

has a full reservoir capacity of 8500 acre-feet, the piping failures occurred when the depth of the water in the reservoir had reached only about 15 feet, at which time the volume of water stored was approximately 1070 acre-feet. The initial leak which rapidly led to a piping tunnel had a head loss of approximately 15 feet over a seepage distance of roughly 300 feet, for a gradient of about 5%.

Attributed Failure Causes

As seen in Columns 16 and 17, Exhibit 1, in none of the 11 failed dams was there an obvious cause or any strong evidence and, consequently, there was no unanimity of opinion among the various investigators concerning the probable cause of failure. In 7 of the 11 dams, the presence of dispersive soils was cited as a possible failure cause.

It should be emphasized that while differential settlement cracking was frequently suspected as a cause of failure, in not one of the cases were any differential settlement cracks observed.

Surface Erosion from Rainfall

As seen in Column 18, Exhibit 1, about half of the failed dams have no trouble with surface erosion and the other half have damage from vertical tunnel erosion ("jugs"), which varies from a minor problem to very severe erosion. Probably of the 11 dams, the worst erosion from rain occurred at Leader Middle Clear Boggy Creek Site 33, where the largest vertical erosion tunnel had reached a diameter of about 4 feet just before the dam failed by breaching in 1969.

The experience at Leader Middle Clear Boggy Creek Site 29 is illustrative of many which lead to the conclusion that severe vertical tunnel erosion can occur even though there is an excellent growth of grass. This is a dam in which a local farmer took a special interest and spent considerable effort in fertilizing and cultivating the grass. As a result the dam is covered with a growth of Bermuda grass which would be considered under any standard as "excellent" protection against surface erosion; however, the vertical tunnel erosion was the worst that the writer saw in Oklahoma.

Four dams in Oklahoma have now been covered with a 12-inch layer of

lime treated soil. Two of these (Leader Middle Clear Boggy Creek Site 15 and 33) were treated only after being in operation for several years, during which time severe damage from rainfall erosion occurred. While the oldest of these lime treated surfaces is only two years, all of them are in excellent condition at the present time, in spite of the fact that they do not generally have good growths of protective grass.

MISSISSIPPI AND ARKANSAS PROBLEMS

The SCS has built nearly 400 earth and flood control dams in the state of Mississippi. These are generally homogeneous clay embankments, quite similar to those constructed in Oklahoma. Of these, three have failed by breaching and there is a serious problem of tunnel erosion from rainfall on many of them. The three breaching failures all occurred at the conduits.

Surface Erosion from Rainfall

The problem with rainfall erosion has been limited generally to the geographic area shown in Exhibit 3. Main details of the experience are as follows:

1. The problem has been getting worse in the last few years and as yet no good survey has been made of how many dams are damaged and how badly. It is estimated that 75% of the dams in the area shown in Exhibit 3 are damaged to some extent and that there may be 50 dams which have fairly severe tunnel erosion.

2. The writer visited 7 dams. The average of these had damage from tunnel erosion which was about the same as the worst examples seen on the visits to the Oklahoma dams.

3. The general dimensions of the tunnels and the appearance of the erosion damage is very similar in Mississippi and Oklahoma (and in the Venezuelan Dike). There can be little doubt that these peculiar tunnels originate by rain runoff entering drying cracks. All three areas (Oklahoma, Mississippi and Venezuela) have relatively high annual rainfall (45-60 inches) and all three also have a climate in which very heavy concentrated rainstorms often occur immediately following periods of hot weather without rain.

4. It is estimated that about 20 of the Mississippi dams have been already repaired by contract with earth moving equipment, either by regrading the whole slope or by excavating and filling individual tunnels. This work has not been highly successful. Some of the dams which have had rather extensive reworking develop a new series of erosion tunnels within 2 or 3 years.

5. All of the Mississippi dams visited had an excellent growth of grass, generally Bermuda, which would normally have provided complete erosion protection.

Potacocowa Site #3

The writer had the opportunity to visit and gather information for only one of the Mississippi breached dams, Potacocowa Site #3. The details of the failure were very similar to many of the Oklahoma failures as listed in Exhibit 1 and described earlier herein. Some of the main features were as follows:

1. The failure occurred during construction when the reservoir reached an elevation of about 4 feet above the invert, at the upstream end, of the reinforced concrcte conduit passing through the base of the dam. Photos 21 and 22, Appendix J, show the condition of the dam soon after the failure.

2. There was no eye-witness to the failure and no good agreement among the various investigators concerning the cause.

3. After construction the dam has developed many large vertical erosion tunnels from rainfall, on both the upstream and downstream slopes.

Geology and Embankment Soil Types

From the standpoint of the normal soil mechanics classification tests, and from the visual appearance, all the soil samples taken from the Arkansas and Mississippi Dams are very similar. These are clays of low to medium plasticity. Clays with apparently similar properties occur very widely in nature, and they are used often for the construction of earth dams.

The situation in Mississippi is different from that in Oklahoma in two major features. First, the embankment clay is a weathered loessial

deposit whereas the Oklahoma soils are residual and alluvial soils which have been derived from the shale-sandstone bedrock. Second, there is no hard rock generally in the foundations of the dams. The abutments and foundations are comprised either of the loess or of an underlying older alluvium. Consequently, there is a much lesser tendency for differential foundation settlement on the average than for the Oklahoma dams, where the abutments are generally hard rock.

Arkansas Problem

The SCS has constructed approximately 120 flood control dams in Arkansas. There have been no failures by breaching and only four dams have been damaged by vertical tunnel erosion from rainfall. These are located within a few miles of each other in the small area shown in Exhibit 3.

The surface soil in this area is a capping of loess underlain again by an older alluvial deposit, so that the subsoil conditions are very similar to those in Mississippi.

The severity of the damage in terms of diameter and spacing of erosion tunnels, is considerably less than the average observed by the writer in Mississippi.

LABORATORY DISPERSION TESTS

For nearly 30 years, the SCS Soil Mechanics Laboratory has employed a laboratory dispersion test (Volk, 1937), with procedure as given in Appendix H. Criteria have been established to identify highly dispersive and erodible clays and designers have been cautioned of the hazards of using these clays in earth embankments, channels and canals.

As a part of this test the content of 5 micron size particles is measured in the standard "hydrometer test", in which a chemical dispersant is used in the water and the soil sample is dispersed in the water with strong mechanical agitation. A parallel test is made in which no chemical is used in the water and in which the soil is not dispersed with strong or longtime mechanical agitation. The content of 5 micron size which goes into suspension in the water in this latter test is compared with the content of 5 micron size as determined in the Standard hydrometer test. For very

erodable clayey soils it is found that the content of 5 micron size measured in the two tests is the same (100% dispersion) and for the typical non-erosive clayey soils it is found that little or no 5 micron size goes into suspension in the second test (0% dispersion).

Reproducibility of Test Results

In this investigationtwo series of laboratory dispersion tests were performed on each specimen. As described in the footnote on the sheets of tabulated test results recorded in Appendix C, one set of tests was performed using a water which had been run through a laboratory demineralizer, and the second set of tests was performed using distilled water. The two series of tests were run at different times. The purpose of performing two tests on each specimen were: (1) to investigate the reproducibility of the test results; and (2) to investigate the influence of small differences in the water purity.

As seen in Appendix C, the results of these laboratory dispersion tests are remarkably consistent. With few exceptions the results of the two tests are very close. Also, with few exceptions the results of the tests using distilled water are slightly higher.

From these results it can be concluded without question that, when starting with the same sample in the laboratory at least, and using a relatively pure demineralized water this is a reliably reproducible test.

Oklahoma Dam Samples

The samples taken during this investigation from the Oklahoma dams which failed by breaching (Exhibit 1) are marked S-1 through S-45.

From the results of these tests it is seen that at least one sample taken from each of the dams had a very high reaction to the dispersion test (60% dispersion or more) and at least one sample from more than half of the dams had 80% dispersion or more. Hence, even with the relatively small number of samples chosen randomly one or more of the test results indicated a sample of very high dispersibility with this test procedure.

The above results contrast to some degree with the dispersion test results summarized in Exhibit 1 where in a few cases many tests were run previous to construction without having one result as high as 60% dispersion. As discussed below it is believed that this is another indication that the results of the laboratory dispersion tests are influenced by the purity of the water employed in the laboratory and/or the natural water content of the sample to an appreciable degree.

These test results also confirm the conclusion that soils which have high reaction to the laboratory dispersion test may often exist in thin layers in close juxtaposition with other soils which have exactly the same appearance and identical index properties. Examples of this can be seen from the test results from almost all the individual Oklahoma dams. One example is a comparison of the test results for Samples S-6 and S-7.1 from Caney Coon Creek Site #2. These two samples were taken from locations very close together and would be considered almost identical materials from every standpoint of soil mechanics classification tests and visual appearance; however, the test results for Sample S-6 show high dispersion and for Sample S-7.1 show low dispersion. Another extreme example is given by comparison of the test results of Samples S-14 and S-16. From the standpoint of visual appearance and from the standard soil mechanics identification tests (Atterberg Limits and grain size distribution) these are identical materials; however, Sample S-14 had 24% dispersion whereas S-16 had 96%. As will be discussed later, these differences usually can be readily explained by the differences in the results of the chemical tests.

Many other similar examples clearly support the conclusions that: (1) within borrow pits and embankments, the dispersibility of the Oklahoma soils varies greatly over short distances; (2) the soils of high and low dispersibility may have identical appearance and identical properties when measured with the conventional soil mechanics civil engineering tests.

The latter conclusion is not limited to clays within general regions of dispersible soils, such as in Oklahoma. For example, the clay of the Cedar Springs Dam, Samples S-95 and 96, is probably the least dispersive

material included in the current study from every standpoint. The Atterberg Limits and grain size distribution of the Cedar Springs clay are indistinguishable for all practical purposes from some of the highly dispersive Oklahoma clays, Exhibit 4.

Venezuelan Dike Samples

As discussed in Appendix B and C, Samples S-46 through S-51 were taken from a section of the clay flood control dike in a zone of bad erosion and Samples S-52 through S-57 were taken from a section of the dike where there was little or no erosion. If we throw out the extreme test result in each of the groups we get a fairly good differentiation between the two zones of the dike as follows:

Eroded Section of Dike	60 to 91% Dispersion
Non-eroded Section of Dike	19 to 58% Dispersion

Hence, from this data alone it appears that 60% dispersion represents a threshhold, above which clay dams may be damaged by heavy rainfall.

Mississippi and Arkansas Dam Samples

All the samples from these sites were taken from the embankments directly adjacent to or in the immediate vicinity of erosion tunnels from rainfall.

All the Arkansas samples (Samples S-76.1 through S-79) have fairly high dispersion--52% to 100%, generally confirming the results of the Venezuelan tests above.

Four of the seven tests on the Mississippi samples (S-85 through S-91) showed dispersion in the range between 22 and 47%. The other three samples, which were all from Big Sand Creek Site 8, had high dispersion (88 to 95%). For these samples with low dispersion there was a relatively greater difference in the test results caused by the distilled water, compared with the demineralized water, than in any of the other samples included in the study. For example, for Sample S-85 a test in demineralized water gave 22% dispersion and in distilled water 78%.

Big Sand Creek Site 8 is one of the Mississippi dams which have been

most badly damaged by surface erosion from rainfall (see Photos 3, 4 and 5, Appendix J). It is planned to repair this dam in 1971 by covering the slopes with a layer of gravelly soil, probably at a cost of the order of $75,000. In connection with studies of the repairs of this dam a number of samples were taken from the embankment and tested prior to the present investigation (Lincoln SCS, 1971). The dispersion tests results ranged between 13 and 24% in contrast to the values of 86 to 95% obtained in the current investigation (Samples S-87 through S-89). There is little doubt that the soil types tested in the two programs were the same, on the average, for practical purposes; therefore, there is apparently something in the sampling-testing procedure which is causing large differences in the dispersion test results for these soil samples.

Samples from Control Dams

The test results on the dams included in the study as "controls" gave generally low results in the dispersion test. Lake Yosemite Dam (Samples S-72 and S-73), was chosen as a very old dam which had had seepage through it for years without piping. The dispersion test results for both samples were both 10%.

Similarly the two samples of the Cedar Springs Dam (Samples S-95 and S-96) were included as a control because the clay core material had been chosen by the designers as a material which probably had high resistance to erosion (Appendix B). The dispersion test results for two samples were 6 and 9% respectively.

Sample S-74 is the Oklahoma clay treated with about 3% hydrated lime on the surface of Frogville Creek Site #2 dam and had a dispersion test result of 11%.

The samples from the Millsite Dam in Utah (S-69 through S-71) represent one of the few essentially cohesionless soils included in the study and is a material which was considered a good core material for a recent dam with no previous knowledge of its dispersibility. The dispersion test results ranged from 16 to 23%.

The above results confirm the general SCS experience (from a great

volume of dispersion testing over the past 30 years), which shows that ordinary fine-grained soils with no appreciable tendency for dispersion piping generally have less than 20 or 25% dispersion in the standard test and that 10% or less is obtained for highly resistant materials.

SOIL CHEMISTRY TESTS

As described in Appendix A, the recent Australian researches have shown that the clay chemistry, and particularly the content of sodium cations on the clay exchange complex and in the pore water, has a major influence on the susceptibility of clayey soils to dispersion piping. The results of this investigation confirm this general conclusion.

Further, the test results performed in this investigation show that, in addition to the percentage of sodium, the total content of salt in the pore water also has an important influence on dispersibility. For soils with very low total salt content, a lesser sodium percentage is required to cause the soil to be dispersive than for soils with a normal or higher salt content.

Description of Tests

For the civil engineering reader it is desirable to describe briefly the test procedures which have been employed. These are relatively standard and routine tests of the agronomist and soil chemist, but are not generally familiar to the civil engineer at the present time.

The tests, whose results are summarized in the tables of Appendix D, are in two main parts. The first part is a set of tests made on the pore water. For these a sample of the soil at the natural water content is mixed with distilled water until the consistency is approximately that of the Atterberg Liquid Limit (at which point the soil scientist refers to "saturated soil paste"). This is allowed to sit for a number of hours until equilibrium takes place between the salts in the pore water and the cations on the cation exchange complex. Subsequently a small quantity of the pore water is extracted from the saturated soil paste (10 to 25 milliliters). At the Lincoln Soil Survey Laboratory this is done with a vacuum and a filter and in other laboratories pressure with a filter is used. This "saturation extract", which is dilute pore water, is then tested by water testing procedures employed by the chemist to determine the amounts of the main metallic cations in solution (calcium, magnesium, sodium and

potassium), in terms of milliequivalents per liter. The electric conductivity of the saturation extract is also measured, which is known to be fairly closely related to the total salt content, and the pH. From the results of these tests the sodium adsorption ratio (SAR) is calculated by the formula:

$$SAR = \frac{Na}{\sqrt{\frac{Ca + Mg}{2}}}$$

where Na, Ca and Mg are in milliequivalents per liter of saturation extract. Hence, the SAR is obtained only from measurements of the dissolved salts in the saturation extract and is a function of both the total salt content and the relative proportions of calcium, magnesium and sodium cations in the water.

The second type of test is independent of the pore water and is the measure of the total cation exchange capacity (CEC) of the clay in terms of milliequivalents per hundred grams of dry soil. Also measured are the relative amounts of the four basic cations on the "exchange complex", also in terms of milliequivalents per hundred grams of dry soil. The results of these tests are frequently presented in the form of the exchangeable sodium percentage (ESP) which is computed by the formula:

$$ESP = \frac{(Na)}{CEC}$$

where (Na) and CEC are in terms of milliequivalents per 100 grams of dry soil.

<u>ESP and SAR</u>

As discussed above the ESP and SAR are both measures of the relative amount of sodium in the soil. Theoretically they are related and it has been shown that for most soils there is a fairly good correlation between them (Richards, 1954). Most of the Australian researches have concluded that it is sufficient to test only the saturation extract and to estimate the ESP. Exhibit 5 shows a plot of ESP vs SAR for the soil samples taken from the Oklahoma Dams. From this it is seen that, with some scatter, the points follow fairly well the equilibrium curve determined by Richards, 1954. From Exhibit 5 it can also be seen that the ESP for the Oklahoma samples ranges from 1 to more than 30 with many test results of 10 to 15 or more, which is the threshhold determined by the Australian investigators over which clays are erosion susceptible (Appendix A).

As seen in the upper graph of Exhibit 6, the samples from the Mississippi, Venezuela and Arkansas dams have essentially no ESP values above 15, but

the median value is about 7.

In the lower graph of Exhibit 6, which includes all the samples from the other dams, it is seen that there are no ESP values above 10, and the median value is less than 2.

Because of the general relationship between SAR and ESP as shown in Exhibits 5 and 6, and because all other correlations have a considerable scatter in any event, the writer concludes that it would not have been necessary to determine the exchangeable cations or compute the ESP. This confirms the general conclusion of the previous investigators that it is sufficient for studies of clay dispersibility to carry out all the testing on the saturation extract.

Comparison with "Nonsaline-Alkali" Soils

It is well known by agronomists that both the salt content in the pore water of the soil and the percentage of the salt which is sodium are important properties governing the behavior of the soil surface from the standpoint of agriculture. Because of their earlier studies of this subject, it is interesting to compare the results of this investigation with the categories of soil groups established by the soil scientist. In the practice of the U. S. Department of Agriculture, the total salt content is commonly measured simply by measuring the electrical conductivity of the saturation extract in millimhos per centimeter (mmhos/cm), and the effect of salt on the yield of crops is given roughly by the following relationship (Richards, 1954).

Conductivity of Saturation Extract (mmhos/cm)	Influence on Crops
0 to 2	Salinity has negligible effect
2 to 4	The yields of some very sensitive crops may be restricted.
4 to 8	Yields of many crops restricted.
8 to 16	Only a few tolerant crops yield satisfactorily.
16 and above	Very few crops yield.

In comparison with this classification system, 80% of the samples tested in this investigation had conductivity less than 2 mmhos/cm and none had more than 8 mmhos/cm.

For agricultural purposes fine grained, clayey soils are categorized further in the following general groups:

Normal Soils	ESP less than 15 and conductivity less than 4.0.
Nonsaline-Alkali Soils	Conductivity less than 4.0 and ESP greater than 15.
Saline-Alkali Soils	Conductivity greater than 4.0 and ESP greater than 15.

The nonsaline-alkali soils are particularly unfavorable for agriculture because the clay particles disperse with the result that the ground surface becomes highly impermeable, prohibiting the entry and movement of water. It is seen from this that agronomists have recognized many years ago that soils with ESP in excess of 15 are dispersive. The saline-alkali soils are generally flocculated so that the water can enter, and consequently they are more satisfactory for agriculture.

Exhibits 7 and 8 show summaries of the results of the laboratory tests from this investigation with respect to the above general agricultural classification. Exhibit 7 includes the samples from the Oklahoma, Arkansas, Mississippi and Venezuela dams and Exhibit 8 contains the results of all the other dams. These exhibits show a strong correlation in the data when plotted in this fashion:

1. Almost all the soil samples which classify as "nonsaline-alkali" gave high dispersibility in the laboratory dispersion tests.

2. The dashed curve separating soils of high and low dispersibility runs through the point defined by conductivity equals 4.0 and ESP equals 15, which is the well recognized agricultural criterion as discussed above.

3. Samples with conductivity greater than 4 generally had low dispersibility.

4. The great majority of soils included in the study have ESP less

than 15 and conductivity less than 4.0 mmhos/cm. In this group there is a definite boundary as shown by the curved line above which the soil samples were generally dispersive and below which they were nondispersive.

5. For soils with low saturation extract conductivity, it is not necessary to have high ESP for the sample to have high dispersibility in the SCS dispersion test.

The plots of Exhibits 7 and 8 show that none of the soils tested with saturation extract conductivity less than about 0.5 mmhos/cm had high ESP (or SAR). By examination of a large volume of other test results, it was found that this is a general fact; that is, that there are hardly any soils in nature which have very low salt content and have high ESP. This results from the strong preference of clays for calcium and magnesium cations over sodium. In this regard, the plot of Exhibit 10 is instructive. This shows that by definition of SAR, when the total salt content is low, the SAR must be very low unless there are no Ca or Mg cations in the water. For example, as shown in Exhibit 10, in the circumstance where the salt content in the saturation extract is 1.0 meq./liter, in order to have SAR = 5, the salts in the saturation extract must be comprised of about 98% sodium cations.

SAR as Measure of Dispersibility

Exhibit 9 shows the relationship between the SAR and the results of the SCS laboratory dispersion test for all samples taken from the damaged dams in Oklahoma, Venezuela, Arkansas and Mississippi. This shows a vague relationship but it is clear from this plot and from Exhibits 7 and 8, that it is necessary to use two parameters for the saturation extract to obtain a reasonable correlation with the results of the SCS laboratory dispersion tests; that is, (1) a measure of the relative amount of sodium; and (2) a measure of the total salt content.

Chemical Test Results

The laboratory test results as plotted in Exhibits 7 and 8 show that there is relatively strong correlation between the results of the laboratory dispersion tests and those of the chemical tests. It was found after plotting this data in various forms that one of the most convenient and straightforward correlations is obtained when the sodium content is described simply as an arithmetic percentage of the content of sodium cations divided by the sum of all the base cations.

Exhibit 11[1)] shows the results of tests on the saturation extract of all samples taken from dams which either failed by breaching or were damaged by rainfall erosion tunnels. Exhibit 12[1)] shows a similar plot with summarizing the results of tests on samples of clay dams, carried along in the investigations as "controls". Exhibit 13[1)] is the same plot including only samples which were taken from dams which were damaged by rainfall erosion tunnels (jugs).

As seen in these drawings there is a very strong correlation between the results of the chemical and the laboratory dispersion tests, and between these test results and the dam performance:

1) Exhibit 11 shows that the test results for the great majority of the samples from the failed or damaged dams plot above the curved solid line, and all the samples with more than 67% dispersion are above this line.

2) Exhibit 12 shows that the test results from essentially all of the control samples fall below the curved dotted line and all of these, with one exception, have less than 33% dispersion.

3) The higher the total salt content in the saturation extract, the more resistant the clay is to dispersion piping; e.g., clays with total salt content of 1.0 meq/liter in the saturation extract may be dispersive with 50% sodium, whereas clays with 20 or 30 meq/liter total dissolved salt appear to need 75 to 80% sodium to be highly dispersive.

4) Exhibit 13 shows a similar plot including only samples from dams damaged by rain. These include all the samples from the Mississippi and

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1) Exhibits 11-13 include the test results on the samples listed in Appendices C and D, plus tests on samples from a few additional dams which were tested subsequent to the first main series of tests.

Arkansas dams, since these samples were taken directly adjacent to erosion tunnels. Exhibit 12 includes only those Oklahoma samples taken directly adjacent to erosion tunnels. As seen in Exhibit 18, the samples from rainfall erosion tunnels were all clays of low to medium plasticity (CL).

Exhibit 14 shows a summary of the correlations obtained in Exhibits 11-13, presented in the form of an arithmetic plot.

Exhibits 15 and 16 show the same graph as Exhibit 14 with some of the test results from the individual dams. The upper graph in Exhibit 15 shows the contrast between the test results from the eroded and non-eroded sections of the Venezuelan dike (from each group one extreme sample has been excluded). This plot shows clearly that there is a great difference in the salt content in the samples from the two sections of dike. The samples in the eroded section all have little salt in the saturation extract (less than 10 meq/liter) whereas the salt content in the non-eroded section of the dike is considerably higher and ranges widely with an average of the order of 35 milliequivalents per liter. The lower plot of Exhibit 15 shows graphically the contrast between two samples taken from Frogville Dam Site 2 in Oklahoma. One of these is the typical highly dispersive clay of the region and the other is a sample of the material taken from the slope of the dam which was treated with about 3% lime 12 months earlier.

The lower plot of Exhibit 15 also shows clearly that the two major "control" dams have test results which plot well below the transition.

TESTS ON OKLAHOMA RESERVOIR WATERS

From the recent studies in Australia and Israel (Appendix A) it was shown that the likelihood of failure of small earth dams was much higher when the reservoir water was pure (low content of dissolved salts). Particularly, many of the failed dams had reservoirs with total dissolved salts of less than 5 milliequivalents per liter.

In this investigation the writer took samples of the reservoir waters at the Oklahoma failed dams in January 1971, and the samples were tested for a total salt content. The results (Samples S-201 and S-219, Appendix D) show that the salt content in all these reservoirs was very low. The salt

content ranged from 0.3 to 2.5 milliequivalents per liter and more than half of the samples showed less than 1.0 milliequivalents per liter. Therefore, while there is no control to show that failure may have been less likely if the reservoir waters had higher salt contents, the results of this investigation are consistent with the conclusions of the previous studies.

CLAY MINERALOGY

Mineralogical (X-ray) analyses were performed on 9 typical clay specimens from Venezuela, Oklahoma and Mississippi. The test results were similar for all samples, showing mixed mineral composition generally with an appreciable content of smectite, the active clay group including montmorillonite. Particularly, it was not possible to see any difference in the clay mineralogy between samples of the Venezuelan dike from the area of bad erosion damage and from the non-eroded area (Appendix D).

The Corps of Engineers' Grenada Dam is located within the general geographic area in Mississippi in which many of the low SCS flood control dams have been badly damaged by rainfall erosion tunnels, and it has developed similar tunnels. As a part of a study of this problem, the Vicksburg District, Army Engineers, sent a series of clay samples to Professor R. E. Grim at the University of Illinois for X-ray and differential thermal analysis. Some of the clay samples were taken directly adjacent to rainfall erosion tunnels and others were taken from areas where no erosion occurred. No consistent or important difference was found in the mineral composition of the two groups of samples. The 10 samples analyzed all had about the same percentage of the main minerals as follows:

Montmorillonite	30-50%
Illite	10-25%
Kaolinite	10-25%
Quartz	20-40%

A number of the Australian investigators have the opinion that all dispersive soils probably have an appreciable percentage of montmorillonite. While not much information was obtained in the current study concerning the mineralogy of the clay, the writer has no reason to doubt this conclusion. It seems likely that most of the dispersive soils taken from damaged dams and tested in this investigation have substantial quantities of montmorillonite. It is possible that the main relationships relating

dispersibility and sodium, as given in Exhibit 14, apply only to soils with at least a moderate content of montmorillonite.

EXCEPTIONS TO CORRELATION OF EXHIBIT 14

In addition to the breaching failures in Oklahoma and Mississippi, samples were taken from two other homogeneous clay dams which failed by piping on the first reservoir filling, the Washington County Dam in Illinois which failed in 1962, and the Stockton Creek Dam in California which failed in 1950 (Appendix B). Both dams failed by breaching with failure details very similar to those of the Oklahoma dams. The Washington County Dam is located in a region where many of the surface soils are known to be highly dispersive (Wilding, 1963). For both dams the chemical test results plotted in the zone of non-dispersive ordinary clays (Zone 3), and the results of the dispersion tests were all low. Hence, from these experiences it must be concluded that either the failure occurred in spite of the fact that the embankment material had the erosion resistance of an ordinary clay or that there may have been localized zones in the embankments of high dispersibility which were not sampled in this investigation.

In addition to the main groups of dams studied, samples were obtained of the core material from four moderately high, zoned dams in various parts of the world which have had trouble with piping in the last few years. As seen in Exhibit 17 the test results for the Balderhead and the Hyttejuvet dams (Appendix B) fall in the lower part of Zone 3 which is the expected region of ordinary, erosion resistant clays. In spite of this, both have one sample with 50% dispersion. Since both of these dams were damaged by piping of fines from the core, it seems reasonable to conclude that the correlation of Exhibit 14 does not apply to the materials used for the cores of these two dams, which were both glacial tills. Hence, if the samples tested for the Balderhead and Hyttejuvet Dams are representative of the core material it would have been impossible to identify them as piping susceptible materials from the results of the chemical tests; however, the results of the SCS Laboratory Dispersion Test would have given an indication that the materials were more susceptible to dispersion than ordinary clays.

Several of the test results for samples of the core of the Hills Creek Dam (Appendix B) also plot in Zone 3 and have 56 and 80% dispersion, Exhibit 17. The clay size fraction in the core material of the Hills Creek Dam are primarily comprised of halloysite. Hence, the samples of the Hills Creek Dam are also exceptions to the correlation of Exhibit 14, in the sense that the material would not have been easily recognized as a dispersive clay from the chemical tests alone.

The admittedly small amount of experience represented by the test results of the Hills Creek, Balderhead, and Hyttejuvet Dams indicates that certain clay types, which may be relatively susceptible to erosion and piping in the core of a dam, cannot be recognized by the chemical tests but can be identified by the SCS Laboratory Dispersion Test. Possibly these clays are of fundamentally different mineralogical composition than the typical clays of the preponderant number of samples tested from the Oklahoma, Mississippi and Venezuelan structures.

The results of the single test on the core material from the Matahina Dam which was badly damaged by piping (Appendix B), a gravelly clay derived by deep in-situ weathering of a greywacke bedrock, are consistent with the main results of this investigation as plotted in Exhibit 14. As seen in Exhibit 17, the material has a sufficiently high percentage of sodium to be included in Zone 1.

RAPID DISPERSION (CRUMB) TEST

As discussed in Appendix E, all samples gathered in this investigation were tested with a rapid dispersion test consisting simply of dropping an air-dried crumb of the soil into a small beaker of water and observing the tendency of the water adjacent to the crumb to become colored by a colloidal cloud of clay particles in suspension.

As described in more detail in Table E-1, Appendix E, four reaction grades were employed:

Grade	Reaction
1	No reaction
2	Slight reaction
3	Moderate reaction
4	Strong reaction

As described in Appendix A, Rallings (1966) concluded from a study of a number of the failed and nonfailed dams in Australia that the results of this rapid dispersion test gave as good a correlation with the performance as any other single indicator. After some experimentation Rallings recommended using a very dilute solution of sodium hydroxide (0.001 normal) rather than pure water as giving a slightly better correlation. As described in Appendix E in this investigation we used demineralized water and sodium hydroxide solutions of both 0.001 and 0.006 normal. As shown in Table E-2 there was not a great difference in results between pure water and 0.001 normal sodium hydroxide; however, the reactions with the 0.006 normal sodium hydroxide were considerably higher in general.

Oklahoma Dam Samples

From the results of the tests on the Oklahoma samples (S-1 through S-45) it is seen that at least one sample from each of these dams exhibited either moderate or strong reaction. Further, with one exception, one sample from each of the dams exhibited a strong reaction to the test.

Hence, even with the relatively small number of samples which were taken in a more or less random fashion from each of these dams, the test results indicate that at least one of the samples had high dispersibility in this test.

Further, with some scatter there is a very strong correlation between the results of the SCS laboratory dispersion test and the rapid (crumb) test. With only a few exceptions the samples which gave moderate or strong reactions in the rapid test were those samples which gave 60 to 100% dispersion in the SCS laboratory dispersion test and the samples

which gave no reaction or slight reaction to the rapid test have low dispersion in the SCS laboratory dispersion test.

At Upper Clear Boggy Creek Site 50 Dam, the most recent of the Oklahoma failures, a considerable number of samples was tested, some of which were taken from borings put down through the embankment directly adjacent to the failure, and others were taken from the walls of the failure breach. Although all the samples were very similar from the standpoint of soil mechanics classification and visual appearance, there was a remarkable difference in the results of the SCS laboratory dispersion test. The results of the rapid (crumb) dispersion test also reflected this difference in dispersibility of these samples and the agreement between the SCS laboratory dispersion test and the rapid dispersion test was remarkable:

Sample No.	Result of SCS Laboratory Dispersion Test	Reaction to Rapid (Crumb) Dispersion Test (0.001 Normal NaOH)
S-14	24% Dispersion	Slight reaction
S-15	30% "	No reaction
S-16	96% "	Strong reaction
S-17	93%	Moderate reaction
S-18	33% "	Slight reaction
S-19	21% "	Slight reaction
S-20	18% "	No reaction
S-21	93% "	Strong reaction
S-22	93% "	Strong reaction

At the Upper Clear Boggy Creek Site 50 it was recognized during the construction of the dam in 1969 and 1970 that the soils were more highly dispersive than had been anticipated by the design studies. A considerable effort was made to try to locate sections of the borrow pit in which the material was less than dispersive and to zone the dam in such a way that the more dispersive material would be placed in less critical sections of the embankment. One of the problems in carrying out this activity was the difficulty in identifying the dispersive material since it's not practical to perform a great number of SCS laboratory dispersion tests in the field during construction. It is apparent from

a comparison of the above results that the rapid dispersion test would have been an extremely useful tool. Literally dozens of tests can be run by a single technician in a period of a few hours with no more equipment than a supply of pure water or sodium hydroxide solution and a number of small glass cups.

Mississippi and Arkansas Dam Samples

All the samples from these sites were taken from the embankments directly adjacent to or in the immediate vicinity of erosion tunnels from rainfall. The results of the rapid dispersion tests on these samples did not consistently give moderate or strong reaction. Of the six samples from Arkansas, two gave moderate reaction and four gave slight reaction (using 0.001 normal sodium hydroxide).

Of the samples from Mississippi one gave strong reaction, one gave moderate reaction, four gave slight reaction and one gave no reaction.

From these results it appears that the rapid dispersion test may not be very useful in identifying the troublesome dispersive soils in Arkansas and Mississippi.

Non-Dispersive Soils

The rapid dispersion test on the samples from dams included in the study as "controls" gave no reaction almost universally. Lake Yosemite Dam was chosen as an old dam which had seepage through it for years without piping. The results of the rapid dispersion tests for both samples were "no reaction".

Similarly the two samples of Cedar Springs Dam (S-95 and S-96) were included as a control because the clay was very tough and believed to be erosion resistant. The results of the rapid dispersion test were "no reaction". It is interesting also that these samples gave no reaction when tested in 0.006 normal sodium hydroxide as well.

Washington County, Illinois

As described earlier the samples of clay from this dam are very similar in visual appearance and in the results of tests of Atterberg

Limits and grain size distribution to the clays in the Oklahoma dams; however, all the samples gave very low reaction to the SCS dispersion test, and the chemical tests showed the clay to be in the non-dispersive category.

As shown in Table E-2, four of the five samples gave "no reaction" and one gave "slight reaction" to the rapid dispersion test.

Summary

This test shows great promise as being a very helpful tool for the soil mechanics engineer. Possibly it will prove more useful in certain areas with certain types of soil than others.

The results of the tests on the Oklahoma dam samples as described above show a very strong correlation with experience (that is, at least some of the samples from each of the failed dams showed a strong reaction) and with the standard SCS laboratory test. In fact, for the Oklahoma dam samples the results of this investigation indicate that the rapid dispersion test correlates equally well with the performance experience of the dams as the results of the SCS Laboratory Dispersion Test.

INITIAL LEAKS CAUSED BY HYDRAULIC FRACTURING

Facts Available

The main pertinent aspects of the breaching failures common to most or all of the experiences are as follows:

1. The depth of the water in the reservoir was not great. In many cases the overall hydraulic gradient causing the leakage flow was less than 15%, and in several cases it was of the order of 5% or less.

2. Both the first leak and the failure occurred within a relatively short time after the reservoir rose rapidly, usually for the first time. The initial leak appeared through the dam within a few days, and often within a few hours, after the rainstorm which raised the reservoir. Hence, the leaks must have been travelling in a concentrated leakage channel, such as a crack. This is true because the dams were all constructed of compacted clays with coefficient of permeability of the order of 0.1 to 1.0 feet per year. It would have been impossible for the leakage water to pass through the embankment in such a short time except through an open leakage channel.

3. Once the leak started the embankment eroded rapidly and tunnels of 10 feet in diameter were frequently eroded in 24 hours.

4. Dams were constructed of clayey soils of low to medium plasticity which have especially high susceptibility to dispersive piping, as determined by all the available testing techniques--laboratory dispersion tests, measures of sodium content and crumb tests.

5. It was not possible to determine conclusively or reliably, by post-construction investigations, that the design or construction of the dams which failed were different in any important or consistent aspect from the general practice.

6. The failure tunnel or breach generally developed at a position where a considerable tendency for cracking due to foundation settlement would be expected.

In investigating any given one of these failures as an individual case it is not possible to be confident that some wholly unknown factor may have been the cause. However, now that it is possible to look backward in time at a group of similar failures, there are a number of other factors pertinent to the evaluation of the failure cause.

1. The 11 failures are located within a relatively small geographic area in Oklahoma.

2. There are many other localized areas of comparable size within the state of Oklahoma in which the SCS has built comparable numbers of dams, of the same general design and construction, in which no failures have occurred.

3. Particularly in western Oklahoma in certain areas the SCS has constructed hundreds of dams without a single failure. In this general area the dam embankments are constructed of soils which show very little or no dispersion in the SCS Laboratory Dispersion Test. These dams have been designed and constructed by the same criteria, by the same people and during the same period of time as the failed dams.

4. In Mississippi, where it is well known that the soils are highly dispersive, because of the peculiar problem of tunnel erosion

from rainfall, there have been three breaching failures, very similar to the Oklahoma failures. The Mississippi dams have been designed and constructed by the same general practice as the Oklahoma dams but by different individual engineers.

5. In other areas of the United STates, such as for example the states of Iowa and Nebraska, the SCS has built many hundreds of clay flood control dams without a single failure. In these areas the clay embankment soils generally have little or no dispersion when tested in the SCS standard laboratory dispersion test.

6. Within the general small area in Oklahoma occupied by the failed dams the SCS has built many other flood control dams similar in design and construction which have not failed. Undoubtedly many of these are constructed of similar highly dispersive clay.

7. The failures have generally been repaired by simply reconstructing the breached section, after which the dams have performed satisfactorily.

From all the evidence, the writer believes it is overwhelmingly probable the following general conclusions are correct:

1. The initial leaks which have led to failure have generally been through fine cracks in the embankment resulting from one or both of two main causes: (1) differential settlement; and (2) drying of the embankment surfacesduring or after construction. The initial leaks may have been very small.

2. The initial leaks caused failure by progressive erosion (piping) because the emabnkments were constructed of highly dispersive clay.

The writer believes it is probable that very small concentrated leaks develop in a considerable percentage of all the homogeneous clay flood control dams constructed. For dams comprised of most ordinary clays, these small leaks aren't capable of eroding the embankment and probably eventually are sealed by

swelling. For dams comprised of highly dispersive clays even a very small leak can cause progressive erosion leading to failure.

In the Australian experiences with farm pond dams, it is considered that some of the failures are initiated by general seepage through the macro-pores in the loosely compacted embankments. At the SCS Oklahoma failed dams, which were constructed with sheepsfoot rollers and moisture-density control, the clay embankments are so impervious that many decades would be required for a particle of water to seep through the typical embankment; hence, the rapid development of the failures can only be explained by leaks flowing through open hydraulic channels, such as cracks.

Hydraulic Fracturing Concept

In the last few years it has been concluded by several investigators in various parts of the world, who have had the problem of studying difficult to explain leaks through well constructed dams, that such concentrated leaks have developed through cracks which were not open before the reservoir rose and which were never seen. These may be existing closed cracks or new cracks which are opened by the pressure of the reservoir acting on the upstream face of the impervious section of the dam. Such cracking can only develop if the minor principal stress acting within the embankment along the path of the potential leak is less than the water pressure. There have been only a few published opinions on this subject (see Vaughan, et al, 1970, and Kjaernsli and Torblaa, 1968); however, it has been fairly widely recognized that such a phenomena can occur and the writer believes it is the most reasonable explanation for a number of recent failures (Sherard, 1972).

In earlier studies of differential settlement cracking, only two extremes have been considered: (1) open cracks developed through the core; or (2) the core remains uncracked. An intermediate condition must also exist in which cracking is imminent. As the embankment is deformed by differential settlement, the minor principal stress in the zone of potential cracking decreases, and may approach zero, providing that the major principal stress is less than the unconfined compressive strength. When the minor principal stress becomes zero (or even negative if the soil can

withstand tensile stress) a crack is imminent and will open if further deformation occurs.

Although the initial crack may be very narrow, perhaps not even visible, water from the reservoir can penetrate it. As a result the stress acting on the plane of the crack is changed abruptly from zero to a compressive stress which would approach the reservoir head, if the crack did not extend through the core. The result is an increase in the crack width because of the readjustment of the embankment to the new stress condition. In this action the water may enter an existing crack which had been previously closed or a new crack may be formed. The term "hydraulic fracturing" has been employed to cover both situations. The internal stress conditions theoretically necessary for hydraulic fracturing can be present on the first reservoir filling or they can develop later, as a result of continuing differential settlement.

Finite Element Analyses

As discussed in Appendix G, a finite element analysis was made employing techniques recently developed at the University of California in which it is assumed that the elastic modulus varies at every point within the dam as a function of the stress level in order to approximate the relationship between stress and strain in a compacted earth embankment.

To the writer's knowledge the calculations shown in Appendix G represent the first effort which has been made to study the stress distribution in a low earth dam constructed of stiff clay. This is a special case of the earth dam embankment in which the stress level due to the weight is generally lower than the unconfined compressive strength so that minor principal stress can easily be zero.

Sketch G-1 shows the model for which the calculations were made; that is, a 40-foot high embankment, an incompressible rock abutment with moderate slope and an alluvial soil foundation.

Sketch G-3 shows contours of computed minor principal stress in the dam for 3 different foundation settlements, with the assumptions:

1. The foundation settlement occurs during construction as soon as the load is applied;

2. The dam is built in eight equal layers of 5 feet thickness each;

3. The dam embankment material is capable of withstanding tensile stress. Case A shows the stress distribution on the assumption that the foundation settlement is negligible. Case B shows the distribution of minor principal stress on the assumption that the foundation settles a maximum of 0.34 feet. The point of maximum settlement is at the foundation level at the left end of the model as shown in Sketch G-4, which presents contours of equal computed settlement corresponding with the computed stresses of Sketch G-3. Case C is the circumstance where the foundation settles a maximum of 1.42 feet. As shown in Sketch G-3 in Case C, a zone of tension develops in the embankment directly over the rock abutment which reaches a maximum magnitude of approximately 0.2 tons per square foot.

Sketch G-5 shows similar results computed on the assumption that the foundation settles 0.34 feet during construction and then continues to settle by varying amounts. Cases D through G show the minor principal stresses for increasing amounts of post-construction foundation settlement and that in Case G, which represents computations for a total foundation settlement of 1.42 feet, there is a large zone of tension extending nearly to the base of the embankment and having a maximum tensile stress of 1.6 tons per square foot.

Sketch G-6 shows the results of a set of computations in every way parallel to those shown in G-5 except that the computations are made on the assumption that the embankment material cannot withstand tensile stress. In these computations the minor principal stress goes to zero only, and the plot of Sketch G-6 shows the shaded zone of zero minor principal stress, "failed in tension". It is seen that the shaded zone in Sketch G-6 is only slightly smaller than the zone of tension of G-5, indicating that the assumption the embankment can take tension did not have a dominant influence on the calculations.

The main result is to show that calculations, using a mathematical

model with dimensions similar to the typical failed Oklahoma dam and reasonable assumptions of the stiffness of the embankment clay indicate substantial zones of tension develop near the abutments. These tension zones occur when the total foundation settlement is of the order 12 to 18 inches. It is, of course, difficult to be confident that the results of such calculations reliably reflect the stresses which develop in an actual embankment; however, it is clear that the results of calculations using the latest modern techniques do not conflict with the "hydraulic fracturing" explanation for the breaching failures.

A main reason for concluding that differential settlement cracks are probably responsible for most of the initial leaks is that the failures have occurred at locations of probable maximum differential settlement. As an example, it is interesting to compare the relative location of the failure at Upper Clear Boggy Site 50 (Sketch F-11 and Photos 15 and 16, Appendix J), with the location of the zone of tension generated in the mathematical model in cases F and G, Sketch G-5. In studying this example it should be kept in mind that Upper Clear Boggy Site 50 Dam has a length of 1500 feet and the dam height is about the same for the full length. Hence, the fact that the breaching failure occurred where the bedrock is at the ground surface on one side of the breach and where there is about 25 feet of alluvium on the other side of the breach, which is exactly the location at which a leak through a differential settlement crack would be expected, must be otherwise attributed to an enormous coincidence.

Embankment Drying

It is frequently necessary to stop construction of a dam for several days during time of hot, dry weather, and it is not unusual for the construction surface to be stopped for weeks. Invariably, shrinkage cracks develop at such times in clay embankments, unless special precautions are taken to prevent drying. Drying and shrinkage occurs also on the slopes of embankment closure sections and in natural clay surfaces exposed by excavation in cutoff trenches, reshaped channel banks and abutments.

When the construction recommences, the surface is often rewetted and scarified. Commonly, however, no investigation is made of the depth of the cracks and they may not be considered important. Probably in most cases the cracks may not be open to any appreciable width; however, the section of embankment influenced by severe drying, which may extend several feet below the surface, probably continues to have relatively low stresses on vertical planes. After construction is completed, the localized embankment section which has been subjected to drying and shrinkage probably continues to constitute a zone of low minor principal stress, where hydraulic fracturing could occur.

The actions of either differential settlement or drying can create a situation where hydraulic fracturing is possible by reducing the minor principal embankment stress to a value less than the reservoir water pressure at the same level. Both actions reduce the minor principal stresses by causing tensile strains to develop. In the case of differential settlement the embankment is deformed, causing the critical section to be strained in tension (stretched), thereby reducing the minor principal stress. In the case of drying the distance between any two points in the dam does not increase, but the volume of the clay-water embankment between the two points is reduced by shrinkage, reducing the minor principal stress.

The writer has observed that dam cracks due to drying and those due to tensile strain caused by differential settlement are frequently related, in the sense that the pattern and location of drying cracks are similar to differential settlement cracks. Sometimes it is difficult to determine whether cracks developing on the crest and slopes of low clay dams are caused by differential settlement or by drying. Undoubteldy, in some cases both actions have contributed. It would be expected that when a clay embankment is caused to shrink by desiccation the drying cracks would open in the direction of the least resistance; that is, if an embankment were undergoing differential settlement and tension cracking was imminent as a result, then embankment volume shrinkage due to drying would undoubtedly accelerate the development of the crack and increase its width. This combined action is responsible for the fact that drying cracks often occur

in low clay dams in the same locations and directions as differential settlement cracks.

In addition to any drying which might occur during construction of the dam causing localized zones of low minimum principal stress, there is evidence from this study that drying of the surface of a low clay dam after construction can also be a main contributing factor. In this regard the writer believes that it is highly significant that the embankment of Cherokee Sandy Creek Site 8A, the only one of the eleven Oklahoma failed dams in which the breaching failure occurred at a point where there was no tendency for differential settlement, was constructed from clay of much higher plasticity and shrinkage properties than the clay in any of the other Oklahoma failed dams (Liquid Limit ranged from 55 to 90%).

In all probability the initial leak leading to the failure of the Cherokee Sandy Creek Site 8A Dam developed in a series of interconnected cracks by hydraulic fracturing through a zone of the embankment where the minor principal stress had been reduced to a low value by drying and shrinkage. In contrast with most of the other projects the dam embankment was completed more than one year before any appreciable quantity of water was stored in the reservoir and it is believed likely that drying of the completed embankment during this period could have had a major influence on the failure. Following failure of the dam, it was observed that there were frequent drying cracks in the embankment extending down to unknown depths, with some of them as wide as 1 to 2 inches at the surface. Subsequent laboratory investigations showed that typical sepcimens of the embankment clay compacted near optimum water content (about 25%) would shrink 10 to 15% of their volume when allowed to air dry from optimum water content to the shrinkage limit.

Investigators of leaks through other low clay dams have concluded that drying cracks were the probable cause. In a well-constructed low homogeneous clay dam in Australia, piping tunnels developed on the first reservoir filling in 1968 at an elevation where the construction surface had been stopped for three months during dry weather, and the investigators concluded that the most probable cause of the leak was drying cracks. At

this dam also the clay was determined to be moderately susceptible to dispersive piping (James and Wickham, 1970).

Another strong piece of evidence supporting the conclusion that drying cracks can lead to piping failure is the widespread damage to the surface of these dispersive clay dams by tunnel erosion from rainfall. There can be no doubt that these surface tunnels are caused by rain run-off seeping into shrinkage cracks and gradually eroding tunnels of roughly circular cross-section.

Several investigators of farm dam failures in Australia concluded that drying cracks were a main cause of the initial leaks leading to piping failure. In his very instructive study, Rallings (1966) concluded that drying cracks were probably responsible for more than half the failures in the dams he studied. In these dams it was commonly observed that several erosion tunnels or breaches occurred in the same dam at the same time, with the tunnels often being about the same elevation. (See Photo 23, Appendix J.) It is believed that drying cracks in a given layer are the most reasonable explanation for this phenomenon.

From the limited evidence it is impossible to draw strong conclusions concerning the relative influence of differential settlement and drying. The writer believes it is possible that a combination of differential settlement and drying have been required to cause failure in the typical Oklahoma clay dam. This could be one of the reasons for the fact that only a few of the dams have failed and many others with apparently similar conditions have not. There is some specific evidence for this tentative conclusion given in the construction records for Upper Clear Boggy Site 50. At this site the construction was started in the fall of 1969 and completed in 1970. The construction records show that the embankment had been constructed up to approximate Elevation 682 at the right end of the dam by the middle of September 1969 and was allowed to sit at this elevation during the winter construction shutdown. As shown in Sketch F-11 the bottom of the breach is approximately at Elevation 680. Hence, the bottom of the breach and particularly the 2-foot diameter piping tunnel observed in the walls of the breach (Photos 15 and 16) were located more or less at the

elevation of the partially completed construction surface, which had ample opportunity to dry and shrink.

Caney Coon Creek Site 2

The record of Caney Coon Site 2 is of especial interest because two independent erosion tunnels developed more than a hundred feet apart, and because the failure was observed by competent eyewitnesses.

The writer believes that the development of two independent tunnels at the same time is extremely strong evidence supporting the main conclusion from this study that the principal contributing cause of the Oklahoma dam failures is the peculiar nature of the clay from which they were constructed. It is not uncommon that small leaks could develop at the toe of a dam on the first filling of a reservoir at two locations; however, the fact that both of these leaks rapidly caused large piping tunnels can only be explained by the existence of a soil which has extremely unusual susceptibility to this kind of failure.

Caney Coon Creek Site 2 was the only one of the failed Oklahoma dams in which a large portion of the embankment was built on one side of the valley in advance. As seen in Sketch F-7 (Appendix F) a portion of the dam was built on the left abutment up to approximate elevation 612 by September 1, 1964, at which time a closure section was left open on the right side of the valley, for the main purpose of allowing additional time for work on the grouting of the right abutment.

This closure section was filled and the embankment completed very rapidly starting on October 21, 1964, with the whole dam being completed on November 15, 1964, or a little more than three weeks later. This rapid construction of the embankment in the closure section would be expected to cause a considerable tendency for arching and longitudinal stress transfer within the embankment, so that zones of low stress in the lower elevations of the closure section would not be surprising. Hence, the development of the two erosion tunnels through the lower portion of the embankment, constructed last as a closure section, is consistent with the differential settlement cracking hypothesis. Also, as seen in Sketch F-7, the lower

portion of second failure tunnel (Tunnel 2, Exhibit F-7) is not very far from the location of the sloping surface of the embankment section which had been completed in advance. This surface had been exposed to drying in the sun for many weeks before it was covered up by the embankment in the closure section, so that it is conceivable that drying cracks also contributed to the conditions which allowed the initial leak for this tunnel.

WISTER DAM

The Wister Dam in southeastern Oklahoma is a fairly large, homogeneous clay dam, about 100 feet high and 5700 feet long, constructed by the Corps of Engineers. It retains a reservoir of 430,000 acre-feet. As summarized briefly in Appendix B, the dam nearly failed by piping on the first reservoir filling in 1949. During the current investigation the details surrounding the leakage and piping problem were reviewed and an inspection was made of the dam.

The downstream slope has an excellent growth of protective grass which is well maintained by permanent personnel at the site. In spite of this, vertical erosion tunnels from rainfall have developed each year, with appearance and dimensions very similar to the frequent rainfall erosion tunnels which developed on the SCS Oklahoma and Mississippi dams. In this investigation four soil samples were taken at widely spaced intervals on the downstream slope of the Wister Dam, from the walls of the rainfall erosion tunnels. All samples were very similar clay of medium plasticity (CL) which was shown to be highly dispersive by both the chemical tests and the SCS Laboratory Dispersion Test, Exhibit 16.

The Wister Dam is located in Oklahoma. It is constructed of the same kind of dispersive clay as the low SCS Oklahoma flood control dams and it was badly damaged by piping on the first reservoir filling in a manner which was very unusual, but similar to the failures of the low SCS flood control dams. Also, frequent rainfall erosion tunnels occur each year. For these reasons it is overwhelmingly probable that the piping damage at the Wister Dam resulted from the same fundamental causes as the failures of the SCS dams; that is, by piping in an embankment of dispersive clay starting with an initial leak caused by hydraulic fracturing.

Since the adoption of modern methods of construction for rolled earth dams, there has been no close precedent for such a large leak and piping condition: -- a more or less horizontal leakage path of 700 feet through a massive clay dam, under a pressure of only a few feet of water and a gradient of the order of 1 in 50. Because of these surprising facts, investigators studying the dam after the failure had difficulty forming a definite opinion concerning the cause. Finally, however, it was generally concluded that the most probable and reasonable explanation was differential settlement cracking (Bertram, 1967, and Casagrande, 1950). This conclusion was reached in spite of the facts that no cracks were observed before or after the piping occurred and that the total settlements were not very large.

The writer considers that the Wister Dam experience is a relatively important part of the total evidence available for analysis of the current problem. This is true because it shows that the same trouble could develop in a dam which is a much larger and more important structure than the SCS flood control dams, and in a dam which was built by a different group of engineers.

INFLUENCE OF CONDUITS

Of the total of 15 piping tunnels or breaches in the 12 low Oklahoma dams, 7 were located at or near the plane of the conduit, Exhibit 1. All three of the Mississippi failures occurred directly adjacent to conduits.

Of the seven Oklahoma dams, two had corrugated sheet metal conduits which had been backfilled on all sides only with compacted earth (Upper Red Rock Sites 42 and 48). The other conduits were of reinforced concrete pipe, provided with cast-in-place concrete cradles.

It is apparent that it is very difficult to compact soil beneath and get a good bond with the lower half of a corrugated metal conduit. Hence, conceivably the failures which occurred along these metal conduits could have been due to an initial leak which developed only because of inability to compact the soil underneath the conduit. Primarily because of difficulty getting a seal in the lower half and because the metal conduits themselves frequently were not watertight, the SCS has abandoned corrugated

metal conduits, except for relatively unimportant and small structures.

Of the piping tunnels which developed near the plane of the conduits, eyewitnesses in two cases observed that the initial leaks developed through the embankment at about mid-height of the dam, many feet above the conduit (Little Wewoka Creek Site 17 and Leader Middle Clear Boggy Creek Site 29, Exhibit 1). While none of the other examples had eyewitnesses, there can be little doubt that the initial leak which led to the failure probably traveled along a path which was generally parallel with the conduit and not very far from it. In one extreme case in Mississippi a piping tunnel developed along the conduit during a storm which occurred before the dam was completed, Photos 21 and 22, Appendix J.

The writer believes that the high percentage of the erosion tunnels which developed along conduits is consistent with and provides other evidence for support of the hydraulic fracturing explanation as the cause of initial leaks. This is true because it would be expected that zones of the embankment immediately adjacent to conduits would have low internal stresses, both from arching and stress transfer caused by differential settlement and from drying and shrinkage.

For the Oklahoma dams it was common practice to place the conduit near one of the abutments. Also, prior to about 1965 a design detail was commonly used in which a trench was excavated into the natural foundation material along the full length of the conduit, with the bottom of the trench extending several feet below the concrete cradle. This trench was then backfilled with compacted earth up to about 6 inches above the top of the conduit. Then, a narrow, vertical-walled trench was excavated into the compacted embankment, the conduit installed, and the concrete cradle placed underneath without forms. All the Oklahoma dams which failed by piping along conduits were constructed in this way. The main purpose of the compacted earth filled trench under the cradle was to provide a uniform support for the conduit.

Since the compacted earth fill under the conduit was generally less compressible than the adjacent natural foundation soil, this design feature created a condition where stress transfer from differential settlement would be expected directly adjacent to the conduit. This is particularly

true because oftentimes the bottom of the trench under the conduit extended down to bedrock. Because of this and also because the reinforced concrete conduit, cutoff collars and cradle create "hard points" within the embankment, it is very probable that the minor principal stresses existing in localized strips of the embankment directly adjacent to the conduit would be lower than the stresses which would have existed if no conduit had been installed. This design detail is no longer considered good practice where conduits are located on compressible soil foundations. In current practice no earth filled trench is placed under the cradle, and the cradle sides are formed with a positive batter.

In addition, drying and shrinkage cracks are especially likely to develop adjacent to conduits because: (1) it was common practice to stop the embankment construction as the conduit was being placed, and (2) backfill around the conduits must be hand placed. Unless special precautions are taken in the embankment construction around the conduit, the embankment in the immediate vicinity of the conduit is generally exposed to potential drying and shrinkage for a longer period of time than the average embankment construction surface.

For these reasons, it would be expected that the minor principal stresses acting within strips of the embankment located directly adjacent to the conduits would be lower than in comparable locations at a distance from the conduits.

With respect to the locations of the failure, the experiences with the low flood control dams in Mississippi are considerably different than the Oklahoma experiences. In Mississippi, all the breaching failures occurred along the conduit, whereas in Oklahoma in approximately half the failures the conduits were not involved. Also, while the evidence is not conclusive, it is believed that on the average the embankment clays comprising Mississippi failed dams are more dispersive and erodible than the clays of the Oklahoma failed dams. The main evidence for this is that more of the Mississippi dams were damaged by rainfall erosion tunnels and the average dam was more severely damaged by rain than in Oklahoma. Hence, there are two apparent differences between the Oklahoma and Mississippi experiences; (1) in Mississippi all the failures occurred along the conduit

whereas in Oklahoma the conduit was not involved in half the failures and (2) there was a greater number and higher percentage of failures in Oklahoma than in Mississippi, in spite of the fact that the Mississippi clay is suspected to be more susceptible to dispersive piping. These apparent differences can be readily explained by the hydraulic fracturing hypothesis. This is true because the abutments for the Oklahoma dams generally consisted of hard bedrock whereas there was no hard bedrock under any of the Mississippi dams. Hence, there was a much greater tendency for differential foundation settlement in the Oklahoma dams. Because of this it would be expected from the hydraulic fracturing hypothesis that there would be a lesser tendency for failure on the average Mississippi dam and also that most likely location of low minor principal stress and hydraulic fracturing would be adjacent to the conduit.

CONSTRUCTION WATER CONTENT

It has been well established in past studies that the likelihood of differential settlement cracking in earth dams is increased when the embankment is constructed with relatively low water content. As a part of this investigation, an effort was made to evaluate the influence of this factor by reviewing the results of the water content tests made during construction of the Oklahoma dams. From this effort it was not possible to arrive at any definite conclusions. The construction records did not show that any of the structures were compacted excessively dry relative to Standard optimum water content (ASTM D698); however, in some of the post-failure studies apparent dry embankment material was observed. Undoubtedly, some sections of some of these dams were placed at a relatively low water content and very possibly this fact contributed to some of the failures.

It is not believed, however, that excessively low construction water content was a major or consistent factor contributing to the average of the failures. A part of the evidence supporting this conclusion is the experience of the Upper Clear Boggy Creek Site 50, the most recent of the Oklahoma failures. For this dam, it was discovered by tests during construction that the embankment clay was more highly susceptible to dispersion than had been realized from the preconstruction investigations. This

problem was carefully studied and everyone was aware that it was necessary to be especially careful with this dam. From a study of the records of the moisture-density control tests during construction it appears that the average water content was fairly uniform and near optimum.

The Wister Dam experience also supports this main conclusion. More than 900 tests of the construction water content were made showing that the average was very close to Standard optimum, and that there was not a great spread in the water conent (U. S. Army Corps of Engineers, 1959).

These experiences indicated that the failures can and did occur in spite of the fact that the embankments were constructed near optimum water content. It can be speculated that the likelihood of failure would have been less if the average construction water content had been several percent above Standard optimum (ASTM D698). The higher the water content the less rigid the embankment material and the lower the unconfined compressive strength, and the less likely the development of embankment zones of low minor principal stress from differential settlement. On the other hand, the higher the average construction moisture content the more precautions which would have to be taken to minimize the undesirable effects of desiccation.

OTHER DAMS IN OKLAHOMA AND MISSISSIPPI

Several hundred other flood control dams were constructed in the same part of Oklahoma as the 11 failed dams during the same period of time, with similar designs and construction. Many of these are constructed of similar dispersive clay. This is also true of the area of highly dispersive clay in Mississippi.

The reason that the fourteen dams failed and the others did not must be attributed to the fact that the conditions were not as severe as in the others. The embankment stresses may not have been reduced to a sufficiently low value to allow an initial leak to develop by hydraulic fracturing. Alternatively, if small initial leaks did develop, the embankment clay forming the walls of the leakage channel may have swelled more rapidly than it eroded, thereby closing off the leak.

In western Oklahoma, where the soils do not have the unusual dispersive

properties, several hundred similar SCS dams were constructed during the same period of time, often from silty sands and sandy silts with little or no cohesion. One of the problems with the dams in this area was that differential settlement cracks sometimes appeared on the crest and slopes. Most of the more serious cracks were filled by mud grouting. None of these dams failed. The difference in performance is undoubtedly due to the difference in soil types. The fine fraction of the soils in western Oklahoma is not dispersive, is not highly susceptible to drying and shrinkage and often does not have the cohesion needed to support the roof of a leakage channel without collapsing.

SUMMARY OF EVIDENCE FOR HYDRAULIC FRACTURING HYPOTHESIS

There is no available means to prove with complete certainty that hydraulic fracturing was the cause of most of the initial leaks. The conclusion is not amenable to being proved with complete confidence. It could not be proved even if a great deal more information were available concerning individual failures. If all of the failures had been observed by competent eyewitnesses, and if the failed dams had been provided with extensive instrumentation, it would still be impossible to determine the cause of the initial leak with certainty. Therefore, it is necessary to draw conclusions on the basis of inference, statistical likelihood and circumstantial evidence.

The main factors leading to the conclusion that it is overwhelmingly probable that the initial leaks were the result of hydraulic fracturing are summarized below.

1. All the failures developed rapidly from an initial leak, which must have been flowing in a concentrated channel, such as a crack.

2. Most of the Oklahoma breaching failures occurred at a location where the foundation bedrock was near the original ground surface, and where the depth to bedrock was changing rapidly.

3. Computations with a mathematical model simulating the typical physical conditions of the average Oklahoma dam, showed that a large zone of tension can be expected at the point where the bedrock is near the surface on one side and the depth to bedrock varies considerably over short distances.

4. The only one of the 11 Oklahoma failed dams for which the failure tunnel did not occur at a point of high potential differential settlement was the one constructed of clay of highest plasticity, with greatest tendency for shrinkage by drying.

5. Because of the relationships between the height of the relatively low dams and the unconfined compressive strength of the clay embankment material, the minor principal stress in the lower portions of the dam can approach zero, or even be in tension. Consequently, there is no doubt that the pressure of the reservoir water can be greater than the embankment stresses acting on vertical planes.

6. The hypothesis readily explains the large percentage of failures which occurred near the conduits.

7. The fact that similar piping developed at the Wister Dam, even under more extreme and difficult-to-explain circumstances, provides evidence that the initial leaks were not caused by some aspects of the design and construction practices employed for the great number of low SCS flood control dams.

8. Finally, there is no other hypothesis available which offers an explanation of the known facts.

<u>Other Causes of Initial Leaks</u>

While the writer believes that it is overwhelmingly probable that the principal cause of the initial leaks has been hydraulic fracturing, for some of the dams it is probable that there were other important contributing factors. In some cases the initial leak from the reservoir to the downstream exit points may have traveled through embankment cracks for part of the distance and found other leakage channels for other parts of the path. For example, at the Caney Coon Creek Site 2 (Exhibit 1) the right abutment was comprised of a rock formation which had open cracks and fissures. The post-failure study showed clearly that the main erosion tunnel at the right abutment was located just above the rock surface and that some of the eroded clay had been carried by the seeping water into the rock cracks. Hence, the initial leak may have travelled through the rock cracks for part of the length.

The failure of Leader Middle Clear Boggy Creek Site 33 Dam was

different from the average experience in several main aspects: (1) the failure occurred approximately five years after the construction even though the reservoir had been nearly full several times during the five years; (2) two independent tunnels developed, apparently at the same time about 90 feet apart; and (3) the embankment slopes had been very severely damaged by frequent vertical rainfall erosion tunnels, with the worst damage located near the point where the dam finally breached. From these facts it seems probable that the rainfall erosion tunnels contributed to the failure. The initial leakage water may have entered one of the erosion tunnels, then traveled through a fissure opened by hydraulic fracturing, and finally exited through an erosion tunnel on the downstream face. Conceivably also the gradual erosion of the rainfall tunnels could have reduced the average minor principal stress acting on potential leakage surfaces through the dam, increasing the likelihood of hydraulic fracturing.

Owl Creek Site 7 Dam is another example which makes it clear that the initial leak did not have one single cause. As described briefly in Exhibit 1, it was found that the leaking water entered the natural ground forming the right abutment about 80 feet upstream from the upstream toe and traveled within the natural ground until it hit the compacted clay cutoff under the dam. Then the erosion tunnel turned about 90 degrees and traveled along the upstream face of the foundation cutoff for about 60 feet where it encountered the outlet conduit, after which the tunnel traveled parallel with the conduit to the downstream toe. In this case it was apparent that the upstream leg of the leak developed along a crack in the fractured rock of the abutment, which created a favored seepage path.

MECHANICS OF DISPERSIVE CLAY PIPING

The writer has concluded that the breaching failures have been caused by progressive erosion (piping) in embankments of dispersive clay of initial leaks caused by hydraulic fracturing. As discussed above the cause of the initial leaks can never be proved with complete certainty; however, there is no doubt that the peculiar erosion susceptible properties of the dispersive clays played an important role in causing the failures.

Literally thousands of homogeneous dams, with heights of 20 to 50 feet, have been built all over the world. There can be little doubt that many of these have been subjected to foundation settlement and drying, and to a tendency for hydraulic fracturing, of generally similar magnitude and severity to the conditions which existed at the Oklahoma and Mississippi dams which failed. Only few of these thousands of dams have failed, and this general experience has supported the conclusion that it is reasonable practice to build low dams as homogeneous embankments of clay.

The writer speculates that it is probable that the main reason that there have not been a greater number of failures of such dams, by hydraulic fracturing and piping, is that when small concentrated leaks develop by hydraulic fracturing in the dam of ordinary clay, the erosive tendency of the leak is too small to cause piping. Subsequently, the embankment swells slightly, increasing the minor principal stresses and closing the cracks. Probably in most cases this action occurs before any appreciable amount of water seeps through the dam. Therefore, it appears that for small homogeneous dams built of most clays, under the common range of foundation settlement and the common tendency for drying cracks during construction, the embankments are sufficiently self-healing to prevent progressive erosion of leaks through cracks.

The Oklahoma and Mississippi dispersive clays have properties which are sufficiently different than those of ordinary clays, so that this general experience in the profession with the performance of low homogeneous dams is not completely applicable. For these dispersive clays, rapid turbulent leaks are not needed to cause erosion. The individual clay particles tend to go into suspension even in still water and to stay in suspension. Therefore, even the smallest leak, flowing completely without turbulence, can pick up the clay particles and carry them along. Also, the highly dispersive sodium clays are very impervious and, while they have a fairly high swelling potential, they absorb water and swell at a slower rate than ordinary clays. Hence, it is speculated that in dispersive clay embankments cracks would swell shut more slowly and that there would be more time for erosion to occur. While the conclusion must remain speculative, this relative timing of swelling and erosion of the walls of the leakage channel in the clay embankment may be a vital factor.

It should be pointed out that the nature of the leak erosion (piping) which has caused failure in these dams is fundamentally different than that which commonly occurs in cohesionless soils, as described in soil mechanics reference books (see, for example, Terzaghi and Peck, 1963, pp. 612-615). In the classic explanation of piping in cohesionless soils, the concentrated leak emerging at the downstream side is caused by water flowing through the soil pores. The erosion starts first at the discharge end of the leak, causing a local enlarged, tunnel-like leakage channel, which creates a local concentration of seepage and erosion forces. As the piping progresses toward complete failure the enlarged leakage channel is gradually extended upstream by progressive erosion until it reaches the water source supplying the leak, at which time a rapid catastrophic failure may result.

In the failures of the Oklahoma and Mississippi dams, the initial leak is already traveling through concentrated leakage channel from the moment of its inception, probably in the form of an individual narrow crack, or a zone of the embankment which has a series of small cracks. There can be no appreciable contribution to the volume of the leak from seepage through the pores of the compacted clay embankment. The erosion of the walls of the leakage channel undoubtedly occurs along the whole length at the same time.

An important piece of evidence showing that the clays in Oklahoma and Mississippi are relatively unusual compared to the ordinary clays from which dams are built is given by the peculiar rainfall erosion. Until a year ago, the writer had the opinion, based on 20 years of study of the performance of earth dams, that a good growth of grass would protect any embankment material from rainfall erosion. It is now quite clear from the experiences in Oklahoma and Mississippi that this conclusion is not correct for the more highly dispersive materials in these areas. As described earlier in this report and shown in PHotos 5 through 8, dams built of these higly dispersive soils have suffered severe tunnel erosion from rainfall in spite of some of the best protective grass covers that the writer has seen on the slopes of dams.

These Oklahoma and Mississippi dispersive clays are such unusual

materials that they fall outside the normal experience of the civil engineer specializing in soil mechanics. While soils of this type evidently exist in concentrated areas in various parts of the world (Australia particularly), and the agronomist has known about them for some time, there is almost nothing in the civil engineering literature about them. That this is true is evidenced by the fact that the writer has in the last few months described this problem of vertical tunnel erosion in clay embankments to a number of engineers with long experience dealing with dams and soil mechanics. Few of them have had any similar experience, all expressed surprise about the magnitude of the problem, and some are even strongly skeptical that such a phenomenon could occur until shown photos.

Another piece of evidence for the unusual nature of these clays, not previously mentioned, is the fact that the reservoir water in these areas is commonly highly colored (brown or red) indicating an unusually high content of suspended clay particles, and in many cases the water never clears. Even small farm ponds with very small runoff areas, where the runoff areas are covered with a good grass growth, stand full of coffee-colored water and never clear up.

Another graphic indication of the unusual nature of these soils is the appearance of the erosion pattern which develops in excavated slopes through natural soil formations. As seen in Photos 9 through 12, erosion in these soils creates a condition which is wholly unlike the results of erosion developing in ordinary clayey soils. Very deep narrow channels and tunnels are formed which can only be explained by the peculiar combination of high erodibility and cohesion. As shown in Photo 12, particularly, the erosion pattern in some natural soils resembles very much the appearance of solution channels in soft soluble rocks, except that the process takes place much more rapidly. Even in theory, the suspension of the individual small dispersive clay particles in water is very like the process of dissolving a soluble material, since the clay particle goes into suspension and may have no tendency to settle out of suspension.

DISPERSIVE CLAYS ARE NOT RARE IN NATURE

It is well known that clays have a stronger affinity for divalent calcium and magnesium cations than for monovalent sodium. Because of

this, dispersive sodium clays of Zones 1 and 2, Exhibit 14, are less common in nature than the ordinary clays of Zone 3, which have primarily divalent cations.

It is apparent, however, that dispersive sodium clays are far from rare in nature. In many parts of the world they are relatively common, as shown by the rapidly growing literature on the subject (see References). One piece of evidence for this conclusion is given by collections of laboratory tests of the clay chemistry carried out for agricultural purposes in the United States. For example, SCS (1967) is a summary of several hundreds of tests performed on clay samples taken in various parts of the state of Colorado. This shows that there are clays with very high sodium content, which would fall in Zones 1 and 2 of Exhibit 14, in many locations on the main Colorado plains, both east and west of the Rocky Mountains.

Other evidence supporting the conclusion that dispersive sodium clays are not rare is given by observations of erosion patterns in natural formations. Once having focused attention on the peculiar patterns of surface erosion in dispersive clays, as illustrated in Photos 9-12, Appendix J, the writer is confident of having seen similar erosion of natural clays in many parts of the world. As an extreme example, there can be little doubt that the distinctive erosion pattern characterizing the scenic Badlands of western North Dakota is caused by rainfall erosion in highly dispersive clay.

Twenty years ago the writer carried out an invesitgation of 60 old homogeneous earth dams in the western United States (Sherard, 1953). These included a number of dams which failed by piping on the first reservoir filling and some other clay dams which had had leaks through them for years without piping. Looking back at this investigation now, it appears likely that most of the failed dams consisted of dispersive clays and that a much better understanding of the performance of these old structures could have been made at that time if tests of the clay chemistry and dispersion had been performed.

There are apparently several sources for sodium cations in dispersive clays. In the Oklahoma clays, which are derived from marine sediments, the sodium is probably a remnant from the salt water of the sea in which

the material was deposited in ancient geologic time. Some of the highly dispersive clays in Illinois are formed as the result of the weathering of a loess deposit, in which chemical disintegration of the feldspar produces an excess of sodium cations (Wilding, et al, 1967).

GENERAL CONCLUSIONS CONCERNING DESIGN, CONSTRUCTION AND MAINTENANCE PRACTICES AND DESIRABLE FUTURE STUDIES

It has been concluded that a principal factor responsible for the breaching failures is the highly dispersive nature of the clays from which the dams were constructed, which clay is a relatively unusual material for which the profession has had little previous experience in constructing earth dams. It is further concluded that the initial leaks which have led to failure have generally been through cracks resulting from either, or both, differential settlement and drying, although other causes of leaks undoubtedly have also been present.

From these main conclusions it follows that efforts to improve practice for future similar dams in these general areas of widespread dispersive soils should be directed toward the following general efforts:

1. Efforts to avoid using dispersive soils, in those circumstances where there are both dispersive and non-dispersive soils available.

2. In those circumstances where all the soils in the dam site vicinity are dispersive, efforts to identify and use the least dispersive soils.

3. Efforts to minimize the detrimental influences of differential settlement and drying.

4. Efforts to minimize leakage through dams from other sources.

5. Continuing investigations directed toward a better understanding of the nature of dispersive soils for the purpose of improving methods of identifying and evaluating the relative dispesibility of the natural soils and also for the purpose of investigating methods available for improving the properties of compacted embankments of these materials at low cost.

The writer's main effort in this investigation has been to review and analyze the cause of the failures which have occurred. Continuing studies are needed to evaluate the best specific measures for improving design and construction practice. During the course of the current investigation the writer developed some general opinions concerning design and

construction practices and needed future studies as given in the following sub-headings.

Differential Settlement of Foundation Soils

Differential settlement is probably a frequent cause of the leaks which have led to breaching failures. In general the critical differential settlement has been imposed on the embankment at a point where the bedrock is near the surface on one side and the foundation consists of 20 feet or more of alluvium on the other side.

Where it is necessary to build a dam at a site where the only soils available are highly dispersive clays and where bedrock is near the surface and drops fairly quickly, special design provisions should be considered in this section of the dam. The writer believes that the following general conclusions are reasonable:

1. A differential foundation settlement of the order of 12 to 18 inches over a length of dam of about 100 feet should be considered enough to cause cracks through which leaks may develop, especially if most of the settlement is predicted to take place after construction. This conclusion is based on the results of the calculations summarized in Appendix G, plus the probability that many of the Oklahoma failed dams were subjected to differential foundation settlements of this order of magnitude.

2. If the potential differential foundation settlement is greater than (1) above, special design measures should be used to reduce the hazard of piping.

3. These special design measures might include:

(A). Reduce differential settlement by excavation of some of the compressible foundation alluvium and/or smoothing our and flattening the abutments;

(B). Reduce potential for embankment cracking and hydraulic fracturing by placing embankment at a higher water content than employed in the past; that is, with average water content 2 or 3 percent above Standard optimum;

(C). Reduce erodibility of critical section of the dam embankment by use of specially located non-dispersive soil, or by treatment of a large zone of the embankment with lime;

(D). Control piping with sand chimney drains.

4. The larger the reservoir and the higher the dam, the greater the need for special design provisions at points of maximum differential foundation settlement.

5. The combination of rock abutments, dry foundation soils susceptible to collapse on saturation, and dispersive embankment materials should be avoided if at all possible. In the extreme case it would probably be prudent to abandon such a site.

Filters for Dispersive Clay

All the Oklahoma and Mississippi breaching failures occurred as a result of an initial leak which emerged freely at the downstream side of the embankment with no influence of a filter. Hence, from the records of the dams included in this investigation, no direct information is available from which to judge whether or not sand filters would have had an important influence in controlling the piping.

It has been speculated by some of the Australian investigators that the individual particles of clay carried in suspension, in the water of a piping leak in dispersible clay, are so small (0.1 to 0.01 microns) that they will not be caught in the voids of even a fine sand filter. "One method of construction control often proposed is to provide an inverted filter at points of seepage outflow. However, filters built to conventional rules cannot prevent the passage of a suspension of deflocculated clay." ... "In the authors' opinion, therefore, no reliance should be placed upon filters in these circumstances." (Wood, et al, 1964)

The writer does not believe that the above pessimistic view is justified. If the initial leaks which led to failure of the Oklahoma and Mississippi dams had exited through sand filters, it is speculated that none of the dams may have failed. The reasons for this conclusion are generally as follows:

1. Obviously the very finest colloidal clay particles suspended in the leakage water would be carried at first through the voids of the sand filter.

2. All the clays in the failed dams had a relatively large percentage of silt size particles, in the range between 5 and 100 microns. In order

for an initially small leak to increase in volume rapidly by piping, the silt size particles and fine sands must also be carried out of the dam; however, these particles would not pass through the voids of a conventional sand filter.

3. Therefore, if the initial leak exited through a sand filter and if the dispersed clay particles passed through the filter, the silt size particles would back up behind the filter and remain in the dam.

4. The silt size particles backing up against the filter would tend to keep the leakage channel filled with relatively impervious material so that the volume of the leak could not become very large, and would probably decrease gradually with time.

5. Even if the leak did not gradually disappear because of the accumulation of silt size particles behind the filter, there can be little doubt that the volume of the leak would increase much more slowly because of the existence of the sand filter than it would have without it. This being the case, the compacted clay embankment material forming the walls of the leakage channel has more time to swell and to squeeze off a leakage channel..

For these reasons, the writer has little doubt that filters of fine to medium sand, designed with adequate thickness and confinement so that the filter itself is stable, will be effective in the control of piping in dispersive clays.

The newly improved understanding of dispersive clay piping makes it highly desirable that laboratory investigations be carried out for the purpose of establishing reliable filter design criteria. Laboratory investigations are needed to measure piping quantitatively in compacted specimens of clay with varying chemical properties as a function of the pressure and gradient of flowing water, the pressure acting within the clay specimen, the exit filter conditions, the grain size distribution of the compacted clay specimen and the chemical properties of the piping water.

Conduits

Since 7 of the 15 Oklahoma piping tunnels, and all three of the Mississippi failures, occurred along conduits, it is clear that when embankments are of dispersive clay special consideration should be given to

details of conduit design and construction. As discussed earlier, it is believed that hydraulic fracturing is especially likely to occur in localized embankment zones of low stress directly adjacent to conduits because of both (1) drying during construction; and (2) differential settlement, arching and stress transfer.

The writer believes that the present design criteria, in which all conduits are of concrete pipe with concrete cradle and in which a compacted earth filled trench is not employed under the conduits, are especially desirable in areas of dispersive soils. A trench of compacted material through a compressible soil foundation can create severe local differential settlement.

For dams which must be built of dispersive soils, consideration should be given, even in low homogeneous dams where no other drain is used, to employing a zone of fine sand around the downstream end of the conduit to act as a filter and a drain. Even though it is theoretically possible for piping of a leak to occur in dispersive clay in spite of a fine sand filter, the writer believes that judiciously placed sand filters would probably have an appreciable influence in reducing the likelihood that a leak would get out of control by piping.

For dams built of dispersive clay, consideration should be given to construction of the dam for a considerable distance around the conduit with specially excavated non-dispersive clay, or to treating the clay with lime.

Both design and construction efforts should be made to find means to install conduits, and get them covered up, which will minimize to the greatest extent possible severe localized drying of the clay surrounding the conduit.

There is no specific evidence available from the facts gathered in this investigation from which to judge the probable influence of concrete cutoff collars. On the other hand there can be no doubt that the collars are additional "hard spots" within the critical zone of embankment directly adjacent to the conduit, which can conceivably contribute considerably to the undesirable stress transfer and potential zones of low embankment stress. It should be possible to design flexible cutoff collars which would not be

greatly more expensive, if any, and would be otherwise perfectly satisfactory.

Drying and Shrinkage

In the general practice for construction of earth dams, drying of temporary surfaces during construction has not been given a great deal of consideration. Nominal efforts are commonly made to prevent drying and to rewet or rework dried surfaces, but it has not frequently been seriously considered that drying could have an important deleterious influence on compacted dam embankments.

The results of this study show that drying and shrinkage are probably a major contributary cause for the initial leaks which led to the piping failures. Therefore, for dams constructed of dispersive clay every reasonable precaution should be taken to avoid severe drying during construction. Clay surfaces which become severely dried, both embankments and natural foundation and abutments, should be liberally excavated to remove the desiccated material.

The experience of Cherokee Sandy Creek Site 8A provides evidence that, at least in severe conditions, drying and shrinkage after construction can result in leaks. The embankment material was a relatively unusual clay with high plasticity and shrinkage potential. Because of this experience, clays with properties approaching those of Cherokee Sandy Creek 8A should be considered undesirable for a homogeneous dam without chimney drain. These are clays which combine very high dispersibility and high Atterberg Limits (Liquid Limit 60 to 90).

SCS Laboratory Dispersion Test

This test is one of the best available tools for evaluating the relative degree of dispersiveness of Oklahoma clay soils. For the present its use should be continued as in the past, though it is expected that the value of the test may be improved in the immediate future by further investigations.

Many experiences indicate that for certain soils which are known to be highly erosive, the SCS Laboratory Dispersion Test gives high dispersion under certain circumstances and low dispersion under others. This is particularly true of the Mississippi soils, but is probably also true of

some of the Oklahoma soils. It is important to carry out an investigation to improve the understanding and reliability of this test.

Expert consultants in the field of colloidal clay chemistry, and related specialties, would be of great value in this investigation.

Chemical Tests on Saturation Extract

A main result of this investigation has been to confirm that it is not possible to identify the peculiar clays which have high erosion susceptibility through the use of conventional soil mechanics tests. It has been conclusively shown that clays with identical visual appearance and identical soil mechanics properties may have radically different susceptibility to dispersive erosion.

Another main conclusion from this investigation has been to confirm the general results of the Australian research showing that the clay chemistry is strongly related to the dispersibility. This investigation has also confirmed that chemical tests on the saturation extract are sufficient to define the soil properties. The results should be compared with the experience summarized in Exhibit 14.

These chemical tests are relatively simple and not costly compared with standard soil mechanics tests, and the writer believes that it will be desirable in the future as a routine part of earth dam design to make the chemical tests on the saturation extract. This applies not only to the practice of the SCS but to the practice of the civil engineer in general working with earth dams.

Rapid Dispersion (Crumb) Test

This test is a useful tool for rapid differentiation between non-dispersive and highly dispersive soils. It has the great advantage that a large number of tests can be run rapidly at low cost by field personnel.

The results of the investigation indicate that for Oklahoma soils the rapid dispersion test correlates very well both with dam performance experience and the standard SCS Laboratory Dispersion Test. The test should be adopted as a routine investigation procedure for future dams in Oklahoma and should be used both during the explorations carried out before the design and during construction.

At the present time an investigation is being made of this test at Oklahoma State University under the guidance of Professor Lester W. Reed of the Department of Agronomy. Professor Reed's study was stimulated by the SCS Oklahoma problems. A purpose of this investigation will be to understand the fundamental colloidal activity occurring when the crumb is dropped in the water, and hopefully superior methods of performing and evaluating the results of the crumb test will be found. Because this test shows promise of being very valuable for SCS projects, particularly in Oklahoma, the writer believes it would be desirable to cooperate with and support Professor Reed in this research to the extent needed to assure a thorough understanding is achieved.

It would be desirable at the present time to carry out some additional studies in the Lincoln Soil Mechanics Laboratory leading toward adoption of at least an interim standard method for performing the test, particularly with regard to: (1) the initial water content of the crumb; (2) the method and criteria for judging the reaction to the test; and (3) the character of the water to be employed. Also, it would be desirable in the Lincoln laboratory to perform rapid dispersion tests on many future samples for which standard SCS dispersion tests are routinely performed, in order to provide data for a comparison of the two tests for all soil types.

Construction Control Procedures

In those circumstances where it proves necessary to build a dam at a site where the only soils available for construction are dispersive clays, considerably greater attention is justified to the construction control than that which is needed for the ordinary clay embankment. Specifically, the writer believes that it would be desirable to have a full time inspector continuously present when embankment is being placed at one of these dams, instead of dividing the time of an inspector between several dams which may be under construction at the same time and in the same region.

The primary purpose is to assure continuous attention to all the small details of construction which may not be vital for a low dam constructed of ordinary soil types, and to minimize the likelihood of leaks

from all causes. Especially it is desirable to enforce the specifications concerning moisture control more literally and stringently than would be considered necessary for equivalent dams constructed of ordinary non-dispersive clays.

Borrow Pit Irrigation

For dams which must be constructed of dispersive clays the writer would give strong consideration to requiring borrow pit irrigation as a routine part of the specifications and construction except in those cases where the natural moisture content of the borrow material is already well above optimum. By irrigating with sprinklers in the borrow it should be possible to reduce the likelihood that dry layers will be occasionally placed within these dams.

It is realized that these dispersive clays are often extremely impervious and that the irrigation water will not generally penetrate to great depths rapidly. Nevertheless, even if the surface of the borrow is sprinkled only a few hours before the area is being excavated, or even concurrently with the excavation, the water has a much better opportunity to be absorbed than it does when it is added on the construction surface.

Specifically consideration should be given to changing the section of the standard specifications which reads, "The application of water to the fill material shall be accomplished at the borrow areas insofar as practicable." by deleting the last three words so as to make borrow area irrigation mandatory.

Rainfall Erosion Tunnels (Jugs)

Magnitude and Nature of Problem. The problem in Mississippi is relatively severe and justifies a considerable amount of study to solve or minimize the problem. Many dams have been affected and the total cost of repairs which will be required on the existing dams during the next ten years can easily exceed a million dollars, and could be considerably more. As an example, bids have been requested this summer for repairs to Big Sand Creek Site 8 by recovering the slopes with a gravelly, non-dispersive soil, plus an experimental lime treated section. The low bid was about $80,000. From the writer's brief look at the magnitude of the problem by inspection

of seven dams in the general area, and discussions with local SCS personnel, it seems probable that 10 to 20 other dams, or even a larger number, will need similar repairs in the next few years.

A main conclusion from this study is that the damage by rainfall erosion tunnels is not a simple problem. Undoubtedly the magnitude of the erosion damage can be reduced by a program in which the tunnels are filled frequently while they are still small so that they don't get to be several feet in diameter. On the other hand, the experiences with the Army Engineers' Wister and Grenada Dams, particularly, show clearly that these tunnels continue to develop in spite of excellent grass and a program of continuously excavating and repairing individual tunnels.

Specifically the writer does not believe that "overgrazing" of the dams is an important part of this problem. Some of the most badly damaged dams in both Mississippi and Oklahoma haven't had any cattle on them for a long time and the grass is 18 inches or higher. On these dams the grass is so thick that the erosion tunnels are obscured and it is literally dangerous to walk on the slopes and crest because of the hazard of falling into one of the tunnels.

Maintenance and Repairs. While the Army Engineers has dealt with the problem on Wister and Grenada Dams for a longer period of time than the SCS, their experience hasn't led to a general solution of the problem. They have had the problem only on these two large dams and they have handled it by heavy maintenance. Their problem is fundamentally different because the depth of these erosion tunnels is small compared to the dimensions of the large dam. Hence, for their projects the erosion tunnels are only a nuisance whereas for the smaller SCS dams the tunnel erosion can conceivably contribute to failure. The Army Engineers have permanent maintenance crews and funds.

When highly dispersive clay is mixed with a small amount of hydrated lime (calcium hydroxide), the properties are changed radically. The plasticity index is generally reduced and the shrinkage limit increased. The reaction to the SCS Laboratory Dispersion Test is reduced to nearly zero by 1 or 2% of hydrated lime. One of the actions of the hydrated lime is to provide calcium cations to exchange with the sodium cations in the clay;

however, in addition, the lime causes other chemical reactions (carbonization and pozzolanic reactions) which actually create new minerals and cement individual clay particles together.

In the last few years four of the Oklahoma dams have been repaired by plating the crest and slopes with 12 inches of clay mixed with 2 to 3% hydrated lime. Two of these dams were treated during the original construction, because the partially completed embankments were badly damaged by rainstorms. The other two were treated several years after construction because the rainfall erosion tunnels were especially severe. Hence, the lime treated surfaces were placed on dams where it was known from experience that the embankment material was exceptionally susceptible to damage by rainfall.

The treated clay was mixed with dry powdered lime in the borrow area with a pulvermixer, sprinkled to bring the water content near optimum, and allowed to cure at least four days. The lime treated clay was then transported and placed on the dam surface in two 6-inch layers and partially compacted with complete coverage by the tread of a heavy crawler tractor. The cost of the lime treated soil has been of the order of \$2.50 to \$3.50/c.y. in place on the dam, about half of which represents the cost of the lime and half the cost of processing and placing on the dam.

Although the performance experience with the four dams is still limited (2 to 4 years), the treatment has been remarkably successful. None of the treated dams has developed any erosion tunnels and there has been essentially no gullying or sheet erosion. The lime treatment raises the pH of the soil, with the consequence that it is difficult to get a growth of grass started rapidly. The pH apparently decreases slowly with time, however, and grass can be made to grow after a few months, especially if the slopes are sprinkled. Exhibit 12 shows the results of tests on the saturation extract from a sample of the lime treated surface from one of the Oklahoma dams, approximately 2 years after treatment. Very probably test results of this sample before lime treatment would have plotted in Zone 1, Exhibit 14.

It is speculated that the lime treated surface has been so completely

flocculated so that there is very little tendency for individual clay particles to be detached from the mass and eroded. Also, the treated surface apparently has practically no shrinkage and cracking caused by drying in the sun so that little if any water can penetrate through the treated layer. Also, because of the freedom from drying cracks, the treated surface layer protects the underlying embankment from drying, in the same way that the embankment material would be protected by a thick layer of moist sand. Hence, the water content of the underlying dispersive clay remains constant with no tendency for shrinkage cracks.

It is possible that small percentages of lime can practicably be mixed with the clay and still have a large beneficial influence. Possibly only a fraction of 1% of lime, applied in some uniform fashion such as by spraying in the form of a slurry, would prove to be a practical treatment.

The writer believes that it would be desirable for the SCS to carry out some investigations, both in the field and in the laboratory to study the practicability of employing lime treatment of typical Oklahoma and Mississippi dispersive clays by methods which use relatively small quantities of hydrated lime, such as perhaps 10 pounds per cubic yard of soil, or even less. Conceivably these investigations might demonstrate that it would be practical and economical to treat relatively large zones of dam embankments in the critical areas, such as adjacent to steep rock abutments. These investigations should include laboratory tests of the influence of very small quantities of lime on dispersibility and field tests to evaluate whether it is practical to add very small quantities of lime in a sufficiently uniform fashion. Also, it would be desirable to study the influence of very small quantities of lime on the ESP and dissolved salts in the saturation extract.

Consideration was given to repairs of the Rio Zulia Flood Control Dikes in Venezuela (Appendix B) by plating the surface with lime treated soil; however, investigations showed that the erosion damage was so severe that it was finally decided to excavate a large part of the total volume of the embankment and rebuild, employing clay which was resistant to rainfall erosion damage, following the criteria of Exhibit 14.

The writer has the opinion that it would be justified to carry out some experimentation to determine whether the erosion on the existing dams could be stopped by a process in which the surface was sprayed with a dilute slurry of hydrated lime in water. If a fresh face of this highly erodible soil is sprayed with such a slurry, a cation exchange will take place at least in a thin skin of clay on the face. This would have the influence of creating a skin of calcium clay on the surface, which conceivably could act as an armor plating. Since it is well known from theory that the clay has a preferential power to hold calcium cations, there is no reason theoretically that the calcium clay forming this armor plating should be later transformed again to dispersive sodium clay.

If one of the erosion susceptible dams were sprayed after a period of dry weather weather with a sufficient quantity of lime-water slurry so that all the existing drying cracks and erosion tunnels were saturated with the lime-water slurry, which might be a quantity equivalent to an inch or two of rainfall, it might be found that all the potential erosion surfaces were armor plated and protected against further severe erosion. Also, there is a possibility that there would be enough residual lime left in the various cracks and fissures so that even if new cracks opened this lime would be washed in and protect them.

The writer believes it would be desirable to select one of the Mississippi dams which is obviously starting to erode badly (such as for example, Potacocawa Creek Site 6) and treat an experimental section using details as discussed above. Another potentially instructive low cost investigation would be to carry out such an experiment on an excavated surface in a natural formation of highly dispersive clay. For example, it would be highly instructive to investigate the influence of a simple lime-water surface treatment on the area shown in Photos 11 and 12, Appendix J. This could be done by cutting back the existing eroded face to a smooth plane surface with a bulldozer, allowing the new surface to dry and crack in the sun for several weeks, and then to saturate half of the surface by spraying a lime-water slurry. The writer believes that there is a reasonable chance that the unprotected portion would be eroded in the next 2 or 3 years back to a condition approaching that shown in the photos and

the sprayed surface would erode much less. Such experiments would also have potential value for minimizing erosion of excavated natural soil faces in canal and channel walls, which has been another SCS problem in Oklahoma, and elsewhere.

Until or unless investigations such as described above demonstrate that there is some easy way to maintain these dams in good condition, it would be highly desirable to maintain them by conventional means, with greater effort than is being employed at the present time. Specifically, the writer has the following opinions:

1. In Mississippi particularly (but also in Oklahoma), it would be desirable to make a detailed survey of all the damaged dams. This should be done by the same engineer, or same group, so that a catalog of relative damage can be compiled based on the same reference framework.

2. In order to prevent excessive deterioration and the need for major repairs at the dams with the worst soils (such as Big Sand Creek Site 8 in Mississippi and Leader Middle Clear Boggy Creek Site 29 in Oklahoma), using conventional procedures, it would be necessary to locate and fill the erosion tunnels at a frequency of at least 3 or 4 times per year. A crew of several men with a small amount of equipment could maintain a considerable number of dams.

Investigations for New Dams. The most economical and simplest design measure for avoiding the problem is to avoid employing the highly susceptible material on the exterior slopes. The main problem is to identify the troublesome materials.

From the results of this investigation we can conclude fairly strongly that not all embankments which have highly dispersive clays on the exterior slopes develop rainfall erosion tunnels. This is shown by the fact that only half of the Oklahoma dams which failed by dispersive erosion had developed surface erosion.

In this regard one of the most instructive individual experiences is the performance of Upper Clear Boggy Creek Site 53. As described in Exhibit 1, the crest and slopes of this dam are in superb condition. There is not the slightest sign of erosion. It should also be mentioned that the grass on this dam is short and that there is every sign that cattle

have been grazing on it continuously. The two samples from this dam were taken in shallow pits excavated in the slopes at about mid-height on the dam, both upstream and downstream (Samples S-12 and S-13). As seen in Appendices C and D both these materials are clays of low plasticity with Atterberg Limits and grain size distribution very similar to the average of the dams which suffered bad damage from rainfall erosion. The test results also show that the soil had high sodium content and is relatively dispersive when measured by the SCS standard Laboratory Dispersion Test and the Rapid Dispersion Test. From the standpoint of the test results the big difference between the samples from Upper Clear Boggy Creek Site 53 and the average of the samples taken from the walls of rainfall erosion tunnels is in the salt content of the pore water extract. As shown in Exhibits 13 and 14 the clay samples taken from the walls of the erosion tunnels all had less than 15 milliequivalents/liter of salt in the saturation extract, and the samples from the worst damaged dams generally had less than 5 milliequivalents per liter. The two samples from Upper Clear Boggy Creek Site 53 had 15 and 104 milliequivalents/liter of salt in the saturation extract, respectively.

From these observations it appears therefore that the soil comprising the surface of Upper Clear Boggy Creek Site 53 is in every way similar to the worst materials in Oklahoma, Mississippi, Arkansas and Venezuela except for the high salt content. As discussed earlier, and illustrated in the upper graph of Exhibit 15, the salt content was also the main test results which differentiated the samples taken from the Venezuelan Dike in the eroded and non-eroded sections. It should be pointed out that this experience is in every way consistent with the fundamental theories of colloidal clay chemistry, in the sense that these theories show that the higher the salt content, the thinner the diffuse double layer around the clay particles and the greater the attraction force between the clay particles.

These observations suggest the possibility (at least theoretically) that the slopes of these dams might be stabilized economically by the addition of some low cost soluble salt other than lime.

From the results of the testing carried out in this investigation it appears that the chemical tests are the only means presently available for

identifying the soils which are susceptible to rainfall tunnel erosion. The results of the standard SCS dispersion test and the rapid (crumb) test might possibly give some indication of the worst materials, but they are not good indicators. For the present it should be considered that the best way to identify the problem materials is to compare the test results with those shown in Exhibits 14 and 18; that is, the most susceptible materials are clays of low to medium plasticity (CL) with liquid limits between 25 and 40, having total salt contents of less than 10 milliequivalents/liter and a sodium percentage in the saturation extract above the "transition" shown in Exhibit 14.

Investigations of Other Chemicals. Treatment of clay with hydrated lime has been used with increasing frequency in the last few years for highway construction. The purpose has been both to increase the ease of construction with highly plastic clays and to improve the strength of the sub-base. Except for the pioneer work by the SCS as described herein, there has been essentially no use of lime treatment to improve the erosion resistance of clays. Lime was chosen for the first efforts in Oklahoma because it is not excessively expensive, it has a radical, beneficial influence on the dispersibility of the clays and because of the precedent for use of lime in the highway industry.

In addition to hydrated lime there are other chemicals which might eventually prove technically and economically superior. Ingles and Wood (1969) propose ferrous sulphate and aluminum sulphate as soluble salts which can be applied as sprayed solutions.

In the United States there is a firm (Iontech, Inc., South San Francisco, California) which is doing very interesting developmental work on clay treatment by ion exchange, largely for landslide stabilization. Their experience indicates that there are many different available chemicals which can be used practically and economically for improving the properties of sodium clay radically. In some applications they have used as little as one or two pounds of dry chemical per cubic yard of treated clay. In addition to chemicals which act simply to provide multiple valent cations to exchange with the sodium, such as ferrous sulphate, they have found that for some clays monovalent salts, such as KCl, have even a more

beneficial influence. In these cases it is speculated that the monovalent salt causes a more complicated chemical reaction with the clay minerals in which free aluminum cations are released in the pore water, causing the clay to flocculate.

The writer believes that it would be highly desirable to investigate the influence of a wide range of available commercial chemicals on the typical Oklahoma and Mississippi dispersive clays. It is easily conceivable that a modest program of relatively simple laboratory investigations would lead to the discovery of an economical and practical means of chemical treatment for the surface of the jugged dams. Possibly also some soluble salt, in a very dilute state, could be used in a spray form during construction of new dams to transform dispersive clays permanently to a non-dispersive, erosion resistant state at moderate cost.

High Hazard Type Dams

Because there is still a great deal to be learned about dispersive soils, the writer believes that for high hazard dams (where a failure could threaten loss of life or large property damage), highly dispersive soils (such as materials which give 50% dispersion in the SCS standard laboratory test) should not be considered suitable for a homogeneous earth dam or for a zoned dam in which erosion of the core could lead to a complete failure.

As background for this conclusion it should be emphasized that the Wister Dam could very well have failed completely if it had not been for the fortunate circumstance that the reservoir dropped below the entrance to the erosion tunnels before the piping could get out of control.

SUMMARY OF MAIN CONCLUSIONS

1. The dams in Mississippi and Oklahoma which failed by breaching or were damaged by rainfall erosion tunnels (jugs) were all constructed of highly dispersive clay.

2. In the same geographic areas as the failed Oklahoma and Mississippi dams there are many homogeneous earth dams built of highly dispersive clay which have not failed.

3. It is overwhelmingly probable that the initial leaks leading to

breaching failures in Oklahoma and Mississippi occurred as a result of hydraulic fracturing.

4. Hydraulic fracturing is believed to have occurred where the minor principal stress acting within the dam on the plane of the potential leak was reduced to a low value, or to tension, as a result of either differential foundation settlement, drying and shrinkage during or after construction, or a combination of these two actions.

5. Most of the failures have occurred in dams constructed of clay of low to medium plasticity (CL and CL-CH); hoever, clays with high liquid limit can also be highly dispersive, and one of the failures occurred in a dam of highly plastic clay (CH).

6. It is not possible to differentiate between highly dispersive clays and ordinary, erosion resistant clays on the basis of visual classification, Atterberg Limits or particle size distribution.

7. In the geographic areas studied, the dispersive clays can be well identified by results of tests of the dissolved salts in the pore water, using standard test procedures of the agricultural soil scientist and the criteria of Exhibit 14.

8. It is speculated that the clays in all the dams included in this study which were damaged by erosion and piping have appreciable content of montmorillonite, and the conclusions summarized in Exhibit 14 may not apply to clays of fundamentally different mineralogical composition.

9. For the soils tested there was an adequately reliable correlation between the exchangeable sodium percentage (ESP) and the results of the tests of dissolved salts in the saturation extract, so that no additional value was obtained by performing the tests necessary to determine the ESP.

10. The SCS Laboratory Dispersion Test is a reliably reproducible test. The results vary somewhat depending on the initial water content of specimen.

11. Results of the SCS Laboratory Dispersion Test give a good measure of the dispersibility of the clays studied in Oklahoma and Venezuela, but gave poor results for the highly dispersive Mississippi clays.

12. Laboratory tests to determine clay dispersibility should be performed as a part of the routine classification testing carried out for

design of earth dams. Both SCS Laboratory Dispersion Tests and tests of the dissolved cations in the saturation extract are generally desirable.

13. The Crumb Test is a very useful rapid dispersion test for the Oklahoma clays, but it is not useful for the Mississippi clays.

14. Dams of soils in Zone 1, Exhibit 14, can be damaged by rainfall erosion tunnels in spite of the very best protective grass cover on the slopes. This is proved beyond doubt by many examples in Oklahoma and Mississippi.

15. The chemistry and dispersibility of clay frequently varies greatly within short distances in apparently uniform deposits.

16. Tunnel erosion due to rain runoff apparently occurs in compacted embankments of dispersive clay only when the total dissolved salts in the saturation extract is less than 15 meq/liter.

17. Stress computations how that tension can be expected to develop in low dams of stiff clay as the result of moderate differential foundation settlement. Post construction settlement of less than one foot, occurring at a point 100 feet away from a rock abutment, causes a large zone of tension in the dam embankment.

18. While damage or failure by piping from initial leaks starting by hydraulic fracturing is far more likely to occur in homogeneous dams of dispersive clay, it is believed that under extreme conditions similar failures have occurred in homogeneous dams of ordinary, non-dispersive clay.

19. It is speculated that sand filters would be effective in controlling leaks safely in dams of dispersive clay.

20. <u>Desirable Further Investigations</u>

The recent developments concerning dispersive clay, initiated primarily by the Australian researchers and extended by the current investigation, have provided important knowledge for the civil engineer building earth dams and related structures. At the same time it is clear that additional investigations are needed to improve understanding of the fundamental relationships, and the criteria and tests employed for design purposes. Particularly the following types of investigations would be potentially fruitful:

A. Laboratory and theoretical studies of the SCS Laboratory Dispersion Test, leading to an understanding of the reasons for the variability of the test results for some soils and for the fact that it does not measure the high dispersibility of the Mississippi clays, and others. Improvement of the procedures of the test or development of a new dispersion test which reliably measures dispersion of all types of clay quantitatively.

B. Studies of various kinds leading to a strengthening and improvement of the criteria shown in Exhibit 14. This can be approached by supplementary investigations of the type included in this study; that is, by taking samples from additional clay dams which were either damaged by piping or display high resistance to erosion. This goal can also be approached in the laboratory by making direct erosion tests on clay samples. One promising means is a technique which has been developed of rotating cylindrical specimens of clay at various velocities in a bath of water. Such tests have been made at the University of Texas and are being currently studied at the University of California at Davis.

C. Studies of the relationship between clay mineralogy and dispersion.

D. Laboratory investigations are needed to study the influence of filters in protecting dispersive clay from piping from concentrated leaks coming through cracks and to develop filter design criteria for soils with dispersive clay fines.

E. Field experiments with lime-water slurries to investigate economical means for reducing the problem of rainfall erosion tunnels (jugging).

F. Laboratory investigations of the influence of other chemicals on the typical Mississippi and Oklahoma dispersive clays could lead to the discovery of means of chemical treatment which are economically and technically superior to lime treatment.

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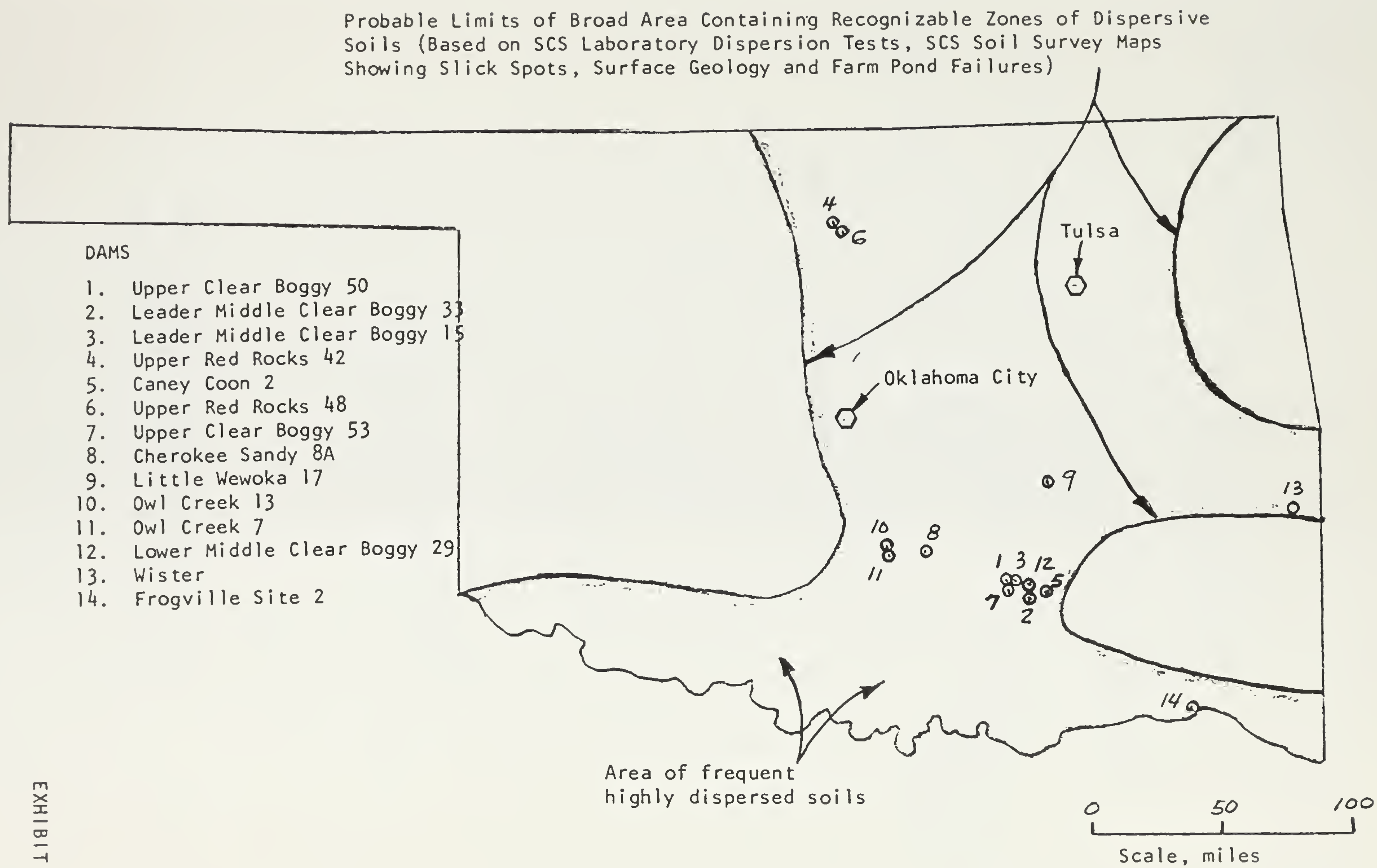
EXHIBITS

VARIOUS INVESTIGATORS ON REPORTS, ETC.) CITED AS PROBABLY CONTRIBUTING OR RESPONSIBLE :king was observed on crest or ny of the dams.)	SURFACE EROSION OBSERVED DURING INSPECTION OF JANUARY 1971 ("Jug" is the local term for vertical tunnels eroded in dam crest and slopes by rain runoff.)
17	18
ferential settlement cracks caused by rapid ying bedrock. 3) Highly dispersive soils. by uplift of right abutment due to swelling	Slopes and crest in fair condition. Grass growth not yet well established. No jugs or deep gullies.
y rainfall.	In past years, both slopes were badly eroded with jugs up to 4 feet diam., especially in area of failure. On repair of dam after failure (Fall 1970), both slopes were covered with lime treated soil. In January 1970, whole dam surface in uniformly excellent condition. No jugs or gullies.
us cause cited. tion carried out in cold and wet weather. ispersive soil. n silty sand layer in foundation. ue to drying or differential settlement hange in depth of bedrock) ispersed soil.	Entire upstream slope was covered with lime treated soil in 1970, because of severe jugging. Downstream slope is badly eroded with frequent jugs to 18 in. diam., in spite of an excellent cover of grass. Maintenance badly needed.
: along side or underneath the conduit, bonding of conduit with bedding or ement of the pipe during backfilling. l the CMP at Upper Red Rock Creek Site bedding or cradle. All other dams in concrete cradle under conduit. Also, Creek Site 48 all other dams had its.)	Operator experienced unusual difficulty getting grass to grow at this dam. Grass cover sparse. No erosion gullies. Moderate jugging with jugs to 12 in. diam. on both slopes, not closely spaced. Condition not serious at present but needs maintenance to prevent rapid worsening.
cracks in rock abutment. 2) Differential ng from steep abutment adjacent or from of natural soil and compacted embankment l settlement cracks resulting from differing bility of foundation soil. 2) Concentrated um. enerally dry of optimum. 2) Highly disper- rs of cohesionless sandy silt in upper fdn.	Good grass cover and slopes in excellent condition. No erosion. Present maintenance is very good. (Dam was completely rebuilt after failure with embankment zoned to put least dispersive soils in outer zones.
l. on dry side of optimum. ally dry.	Fair grass cover and little or no erosion over most of dam surface. A few isolated jugs (6 in. diam. max.) in general vicinity of conduit. Not a severe problem at present but needs maintenance to prevent worsening.
t cracks resulting from deep foundation ailure tunnel and bedrock on the other, nch under conduit. 2) Majority of of optimum. 3) Drying cracks. 4) Highly bond between compacted layers.	Crest and slopes in perfect condition. Not the slightest sign of erosion. Best looking dam seen in this study. Complete grass cover kept short by grazing cattle. Grass a mixture of Bermuda and clover.

DAM NAME	Max. Dam Height Feet (above natural ground)	Reservoir Volume at emergency spillway (acre-ft)	Embankment Soil Properties: Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Grain Size % Finer: #200 sieve	Grain Size % Finer: 0.005 mm	Dispersion % SCS Test: Range of Most Tests	Dispersion % SCS Test: Highest three test results from all samples	DATES: Dam Completed	DATES: First Reservoir Filling	DATES: Failure	FAILURE DESCRIPTION AND DATA (1) Eye witnesses present? (2) Location of initial leak, if known. Best description of progressive erosion. (3) Conduit involved or exposed in failure? (See longitudinal sections for locations of erosion damage, Appendix F.)	FAILURE CAUSE AS DETERMINED BY VARIOUS INVESTIGATORS (DEFICIENCY REPORTS, INSPECTION REPORTS, ETC.): Depth of Bedrock Changes rapidly Near Failure (Yes or No)	Water Depth at Failure Location (Feet)	Approx. Res. Vol. at Failure (Acre-feet)	Failure Cause Definitely Known? (Yes or No)	MAIN FACTORS CITED AS PROBABLY CONTRIBUTING OR RESPONSIBLE (No open cracking was observed on crest or slopes of any of the dams.)	SURFACE EROSION OBSERVED DURING INSPECTION OF JANUARY 1971 ("Jug" is the local term for vertical tunnels eroded in dam crest and slopes by rain runoff.)
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
UPPER CLEAR BOGGY CREEK SITE 50	26	1005	22-35	7-18	60-80	25-35	9-15 pre-const	100, 100, 100 (from 19 tests after failure)	May 1970	Oct. 9 1970	Oct. 15 1970	No eye witness. Failure date not known within 5 or 6 days. Since the bottom of the breach is well above the foundation, it is sure that the failure was wholly confined to the embankment. Conduit was not involved. Vertical walled breach, 50 ft. wide at crest. (See photos 13-16, Appendix J.)	Yes	20	700	No	1) Drying cracks. 2) Differential settlement cracks caused by rapid change in depth of underlying bedrock. 3) Highly dispersive soils. 4) Tension cracks caused by uplift of right abutment due to swelling of stiff clay and shale.	Slopes and crest in fair condition. Grass growth not yet well established. No jugs or deep gullies.
LEADER MIDDLE CLEAR BOGGY CREEK SITE 33	23	370	27-48	11-27	50-75	25-50	0-20	82, 82, 39 (from 17 tests)	Sept. 1963	1964	Oct. 14 1969	No eye witness. It is almost sure that the initial leaks broke through the dam just below the high water level and gradually eroded downward. All the investigators concluded that foundation leakage did not contribute to the failure. Failure occurred following rapid reservoir filling on Oct. 13, 1969. Two independent tunnels developed, apparently at the same time, about 90 feet apart. The larger tunnel was 8 ft. high and 15 ft. wide. The conduit was not involved.	Yes	13	360	No	1) Drying cracks eroded by rainfall.	In past years, both slopes were badly eroded with jugs up to 4 feet diam., especially in area of failure. On repair of dam after failure (Fall 1970), both slopes were covered with lime treated soil. In January 1970, whole dam surface in uniformly excellent condition. No jugs or gullies.
LEADER MIDDLE CLEAR BOGGY CREEK SITE 15	25	270	22-50	7-32	50-80	25-45	20-80	92, 88, 88 (from 17 tests)	June 1965	May 26 1965	5/26/65 (first) 6/1/68 (second)	The dam failed twice in locations 100 feet apart. No eye witnesses to either failure. In the 1965 failure an arch-like tunnel about 3 feet high and 15 feet wide extended through the base of the dam, apparently travelling at the original ground level, passing through or under the 8 ft. deep cutoff trench. In the 1968 failure, a 3 ft. diam. circular tunnel passed through the bottom of the dam. At the entrance and exit, the inverts were 2 ft. and 5 ft. above natural ground, respectively, and the tunnel appeared to be straight through the dam, but could have dipped into the foundation in the central portion of the dam. The conduit was not involved.	Yes	7 (1965)	29 (1965)	No (Both cases)	1965 Failure: 1) No obvious cause cited. 2) Construction carried out in cold and wet weather. 3) Highly dispersive soil. 1968 Failure: 1) Piping in silty sand layer in foundation. 2) Cracks due to drying or differential settlement (rapid change in depth of bedrock) 3) Highly dispersed soil.	Entire upstream slope was covered with lime treated soil in 1970, because of severe jugging. Downstream slope is badly eroded with frequent jugs to 18 in. diam., in spite of an excellent cover of grass. Maintenance badly needed.
UPPER RED ROCK CREEK SITE 42	23	385	28-35	9-18	65-90	20-35	0-20	41, 18, 8 (from 4 tests)	Dec. 1966	June 20 1967	June 20 1967	At 5 p.m., June 20, 1967, a small leak (about 1 in. diam.) was observed emerging just below the bottom of the discharge end of the conduit. No other eye witness observed events until 24 hrs. later when the reservoir was found to be empty from the failure. An erosion tunnel about 3 ft. high and 6 ft. wide average, ran along the right side of the conduit upstream of dam ℄, then crossed over and ran along the left side. The conduit was exposed and washed clean over most of its length, but in certain sections it was still surrounded with intact compacted embankment. The erosion tunnel was confined to the compacted embankment, that is, it did not enter natural soil of foundation.	Yes	5	60	No	1) Piping of initial leak along side or underneath the conduit, resulting from inadequate bonding of conduit with bedding or backfill, or from displacement of the pipe during backfilling. (The 18 in. diam. CMP and the CMP at Upper Red Rock Creek Site 48 did not have concrete bedding or cradle. All other dams in this chart had continuous concrete cradle under conduit. Also, except for Upper Red Rock Creek Site 48 all other dams had reinforced concrete conduits.)	Operator experienced unusual difficulty getting grass to grow at this dam. Grass cover sparse. No erosion gullies. Moderate jugging with jugs to 12 in. diam. on both slopes, not closely spaced. Condition not serious at present but needs maintenance to prevent rapid worsening.
CANEY COON CREEK SITE 2	50	8500	20-32	0-18	60-75	20-30	20-60	91, 79, 73 (from 34 tests)	Nov. 16 1964	Nov. 17 1964	Nov. 19 1964	Eye witnesses present during entire development of the piping failure. Two independent piping tunnels, about 125 ft. apart, appeared at the downstream toe. Tunnel No. 1 was first observed at 10 a.m. on October 19, 1964, with a small leak emerging from a 6 in. dia. area. Apparently travelling at the contact between the dam and rock abutment along the line of the municipal water supply conduit, it rapidly increased in size, reaching 18 in. diam. by 5 p.m., at which time it was carrying an estimated volume of 25 cfs. By morning (Nov. 20, 1964) it was 15 ft. in diameter and about 50-75 cfs. (See photos 19 and 20, Appendix J). The second leak (Tunnel No. 2) appeared at 1:30 p.m. Nov. 20, 1964 and was 10 feet in diameter during next 2 days.	Yes	15	1070	No	Tunnel 1: 1) Piping into cracks in rock abutment. 2) Differential settlement cracks resulting from steep abutment adjacent or from difference in settlement of natural soil and compacted embankment in trench under conduit. Tunnel 2. 1) Differential settlement cracks resulting from differing thicknesses and compressibility of foundation soil. 2) Concentrated leak in foundation alluvium. GENERAL: 1) Compaction generally dry of optimum. 2) Highly dispersive soils. 3) Some layers of cohesionless sandy silt in upper fdn. 4) Leaks independent.	Good grass cover and slopes in excellent condition. No erosion. Present maintenance is very good. (Dam was completely rebuilt after failure with embankment zoned to put least dispersive soils in outer zones.
UPPER RED ROCK CREEK SITE 48	22	394	30-42	12-22	75-85	30-40	10-20	55, 32, 20 (from 15 tests)	Nov. 2 1964	Nov. 16 1964	Nov. 17 1964	No eye witness. Reservoir emptied through erosion tunnel before failure was discovered on Nov. 18, 1964, probably about 24 hrs. after initial leak. Tunnel was about 10 ft. in diameter and located a few feet to right of the conduit. Conduit was exposed and washed clean at both the upstream and downstream ends. During reconstruction it was found that the conduit was completely surrounded by intact compacted embankment over the central 50 feet of its length.	Yes	10	200	No	1) No obvious cause cited. 2) Embankment compacted on dry side of optimum. 3) Foundation soils generally dry.	Fair grass cover and little or no erosion over most of dam surface. A few isolated jugs (6 in. diam. max.) in general vicinity of conduit. Not a severe problem at present but needs maintenance to prevent worsening.
UPPER CLEAR BOGGY CREEK SITE 53	26	360	25-40	12-25	60-90	25-40	20-50	63, 59, 52 (from 19 tests)	Feb. 15 1964	June 16 1964	June 18 1964	No eye witness. Reservoir emptied through erosion tunnel before first inspection about 24 hrs. after initial leak. Tunnel entered the upstream slope at the left side of the conduit, just above its top, then crossed over the conduit and emerged downstream on the right side. Near the dam axis, the tunnel was about 10 ft. in diameter and located with its axis 8 ft. to 12 ft. to the right of conduit ℄. Conduit exposed nearly full length but was not undermined, moved or damaged. Erosion tunnel passed through compacted embankment for full length.	Yes		85	No	1) Differential settlement cracks resulting from deep foundation alluvium on one side of failure tunnel and bedrock on the other, and compacted fill in trench under conduit. 2) Majority of embankment compacted dry of optimum. 3) Drying cracks. 4) Highly dispersed soils. 5) Poor bond between compacted layers.	Crest and slopes in perfect condition. Not the slightest sign of erosion. Best looking dam seen in this study. Complete grass cover kept short by grazing cattle. Grass a mixture of Bermuda and clover.

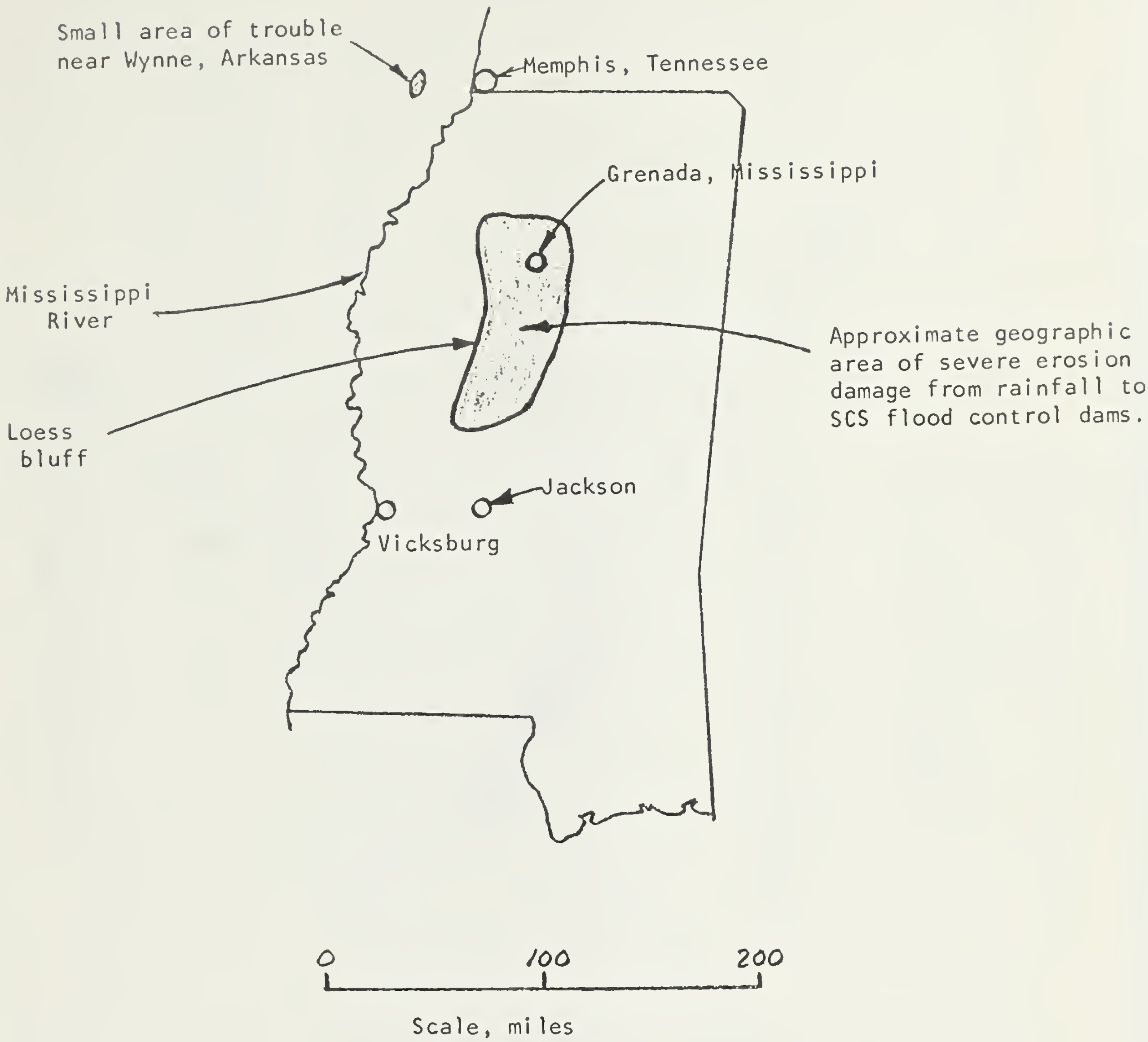
VARIOUS INVESTIGATORS ON REPORTS, ETC.) CITED AS PROBABLY CONTRIBUTING OR RESPONSIBLE cking was observed on crest or ny of the dams.)	SURFACE EROSION OBSERVED DURING INSPECTION OF JANUARY 1971 ("Jug" is the local term for vertical tunnels eroded in dam crest and slopes by rain runoff.)
17	18
unkment and/or foundation during or after ential settlement cracking resulting from of foundation soils (the failure tunnel ion of one of the old stream channels). ing the stiff and highly plastic clay. y.	Crest and slopes in excellent condition. Good grass cover. No erosion gullies or jugs.
ent cracks resulting from rapidly increasing nity of failure, or from difference in atural foundation soil and compacted ench under the conduit. 2) Highly dis- reservoir filling.	Bad erosion, with both jugs and gullies, on the downstream slope near conduit. Also isolated jugs to 12 in. diam. for full length of dam. Erosion has developed in spite of excellent grass growth. (The grass could not be better.) Needs maintenance badly to prevent rapid worsening of condition.
nt cracks resulting from rapidly increasing me material compacted dry of optimum and low bond or backfill compaction around conduit neer, Mr. T. Hayes, who investigated the s was unlikely that the failure was caused onduit). 4) Highly dispersive soils.	No erosion gullies or jugs. Excellent grass cover.
shale of right abutment (possible old itial settlement resulting from difference tural soil foundation and compacted earth 3) Piping in drying cracks. 4) Poor il around conduit.	Excellent grass cover and no erosion over most of the dam. Localized gullies near discharge end of conduit. Not important problem. Minor maintenance needed.
has been made.	Dam slopes and crest are badly damaged by frequent jugs. By far, jugging is more severe than on any of the other dams inspected in this study. This is in spite of an excellent growth of Bermuda grass, which has prevented gullies but has had little beneficial influence on the jugging. So many jugs are on the crest and slopes (generally 6" to 18" dia.) that it is danger- ous to walk on surface. (See photo 7, Appendix J)
l that the leak must have broken through it caused by differential settlement in g from a 30 ft. high shale slope form- e main river channel. This explanation since no cracks were seen on the crest ng to cause differential settlement	In spite of an excellent growth of grass which is mowed and fertilized, they have had a problem with jugs appearing each year for 20 years. They fill them with sand and sod. In Jan. 1971 there were many jugs to 2 ft. or more in diam. The grass cover could not have been thicker or more protective. (See photo 8, Appendix J). The crest is paved with asphaltic concrete and upstream slope has thick blanket of riprap in good condition.

DAM NAME	Max. Dam Height Feet (above natural ground)	Reservoir Volume at emergency spillway (acre-ft)	Embankment Soil Properties: Atterberg Limits: Liquid Limit	Plasticity Index	Grain Size % Finer: #200 sieve	0.005 mm	Dispersion % SCS Test: Range of Most Tests	Highest three test results from all samples	DATES: Dam Completed	First Reservoir Filling	Failure	FAILURE DESCRIPTION AND DATA: (1) Eye witnesses present? (2) Location of initial leak, if known. Best description of progressive erosion. (3) Conduit involved or exposed in failure? (See longitudinal sections for locations of erosion damage, Appendix F.)	FAILURE CAUSE AS DETERMINED BY VARIOUS INVESTIGATORS (DEFICIENCY REPORTS, INSPECTION REPORTS, ETC.): Depth of Bedrock Changes rapidly Near Failure (Yes or No)	Water Depth at Failure Location (Feet)	Approx. Res. Vol. at Failure (Acre-feet)	Failure Cause Definately Known? (Yes or No)	MAIN FACTORS CITED AS PROBABLY CONTRIBUTING OR RESPONSIBLE (No open cracking was observed on crest or slopes of any of the dams.)	SURFACE EROSION OBSERVED DURING INSPECTION OF JANUARY 1971 ("Jug" is the local term for vertical tunnels eroded in dam crest and slopes by rain runoff.)
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
CHEROKEE SANDY CREEK SITE 9A	27	633	55-75	38-52	75-90	30-60	10-100	100, 100, 100 (from 34 tests)	Jan.22 1963	May 6 1964	May 15 1964	Eye witness present from early development of trouble. Initial leak entered upstream slope near high water line, then travelled downward to near original ground surface, then travelled along base of dam horizontally, and then flowed upward again emerging on the downstream slope about 5 ft. above the toe. A tunnel was gradually eroded following the path of this initial leakage, reaching a final diameter of about 8 ft. The reservoir level was at a high point about 48 hrs. before the leak was observed, and another 48 hrs. were required for the release of the main volume of the reservoir through the gradually eroded failure tunnel. The conduit was not involved.	No	17	360	No	1) Drying cracks in embankment and/or foundation during or after construction. 2) Differential settlement cracking resulting from variable compressibility of foundation soils (the failure tunnel was located at the position of one of the old stream channels). 3) Difficulty in compacting the stiff and highly plastic clay. 4) Highly dispersive clay.	Crest and slopes in excellent condition. Good grass cover. No erosion gullies or jugs.
LITTLE WEWOKA CREEK SITE 17	40	2043	20-32	5-17	40-70	10-25	0-100 0-14	100, 50, 33 (from 31 tests before const.) 14, 6, 5 (from 35 tests after failure)	May 14 1960	May 19 1960	May 21 1960	Eye witnesses present from beginning of initial leak. Small initial leak, about 6 in. diam. with "trickle" of water, was discovered at 8 a.m. on the downstream slope on the line of the conduit, about 15 ft. vertically above the conduit. The head on the initial leak must have been 5 ft. or less. By midday the leak had eroded a tunnel 10 ft. in diam. and by evening the reservoir was empty, and the erosion had progressed downwardly until the conduit was exposed. The top of the tunnel collapsed leaving a breach with near vertical walls and width of 47 ft. at the crest. The conduit was exposed full length in bottom of breach.	Yes	17	800	No	1) Differential settlement cracks resulting from rapidly increasing depth of bedrock in vicinity of failure, or from difference in compressibility of the natural foundation soil and compacted soil filling the deep trench under the conduit. 2) Highly dispersed soils. 3) Rapid reservoir filling.	Bad erosion, with both jugs and gullies, on the downstream slope near conduit. Also isolated jugs to 12 in. diam. for full length of dam. Erosion has developed in spite of excellent grass growth. (The grass could not be better.) Needs maintenance badly to prevent rapid worsening of condition.
OWL CREEK SITE 13	24	124	30-40	12-18	60-70	20-40	0-30	63, 43, 42 (from 65 tests)	Jan. 6 1957	May 17 1957	May 19 1957	No eye witness. Failure tunnel travelled along the right side of the conduit. At the dam the tunnel was about 18 ft. in diam. with its bottom near the invert of the conduit. Conduit exposed full length.	Yes	13	36	No	1) Differential settlement cracks resulting from rapidly increasing depth to bedrock. 2) Some material compacted dry of optimum and low density. 3) Inadequate bond or backfill compaction around conduit (though the Project Engineer, Mr. T. Hayes, who investigated the failure concluded that is was unlikely that the failure was caused by the presence of the conduit). 4) Highly dispersive soils. 5) Drying cracks.	No erosion gullies or jugs. Excellent grass cover.
OWL CREEK SITE 7	24	205	25-50	10-25	50-75	25-75	0-30	82, 78, 71 (from 95 tests)	Mar.19 1957	May 17 1957	May 19 1957	No eye witness. Reservoir was emptied by the failure before first inspection on May 20, 1957. The erosion tunnel entered the natural ground of the right abutment about 80 ft. upstream from the dam, travelled in downstream direction until reaching the cutoff under the dam, then turned 90° and travelled parallel and immediately adjacent to the cutoff trench for about 60 ft. where it encountered the outlet conduit, after which the tunnel travelled to the downstream toe, crossing over the conduit from the right to the left side. In the natural ground upstream, the tunnel was about 4 ft. in diam. At the ℄ of the dam, the erosion tunnel was about 15 ft. in diam. and extended 3 ft. below the conduit invert. Conduit was exposed over downstream half.	Yes	16	213	No	1) Piping into cracks in shale of right abutment (possible old landslide). 2) Differential settlement resulting from difference in compressibility of natural soil foundation and compacted earth in trench under conduit. 3) Piping in drying cracks. 4) Poor bond or compaction of soil around conduit.	Excellent grass cover and no erosion over most of the dam. Localized gullies near discharge end of conduit. Not important problem. Minor maintenance needed.
LEADER MIDDLE CLEAR BOGGY CREEK SITE 29	23	258	19-30	2-13	30-70	11-35	17-30	58, 46, 43 (from 8 tests)	Sept.29 1962	1963	1970	Dam has not failed. Since construction the reservoir has been at high level only in 1969 and 1970. In Oct. 1970, at time of high reservoir, a small leak of dirty water was found on the downstream slope coming out of a small hole (about 3" diam.) on the plane of the conduit at an elevation of 10 ft. to 12 ft. above the conduit. After flowing a short time the leak stopped. It is believed that the leak stopped because the reservoir level dropped. On visit of Jan. 1971, it was difficult to distinguish the leakage tunnel from the many erosion jugs on the slope.	Yes	N.A.	N.A.	No	No formal investigation has been made.	Dam slopes and crest are badly damaged by frequent jugs. By far, jugging is more severe than on any of the other dams inspected in this study. This is in spite of an excellent growth of Bermuda grass, which has prevented gullies but has had little beneficial influence on the jugging. So many jugs are on the crest and slopes (generally 6" to 18" dia.) that it is dangerous to walk on surface. (See photo 7, Appendix J)
WISTER DAM (U. S. ARMY CORPS of ENGRS. POTEAU RIVER at WISTER, OKLAHOMA)	100	130,000		7-16	30-90	15-65			Nov. 1949	Jan.23 to 28, 1950	Jan.30 1950	Soon after topping out of dam embankment, heavy rains raised water level for the first time in a few days to Elev. 494 ft. (max. depth about 65 ft.). Two days later (Jan. 30, 1950), a concentrated leak broke through the dam not far below the reservoir level. (The main leaks entered the upstream slope at about Elev. 485 ft. and emerged downstream at about Elev. 475 ft.). The leakage water was obviously very muddy. Estimated volume increased from about 5 cfs to 20 cfs by Feb. 3, 1950. By Feb. 5, 1950, the reservoir was lowered, by discharge through outlet conduits, to Elev. 477 after which the leaks nearly stopped. Subsequently the reservoir was lowered and extensive repairs were made including a sheet pile wall, grouting and berms on both slopes.	Yes	65	260,00[illegible]	No	It was finally concluded that the leak must have broken through a crack in the embankment caused by differential settlement in the dam itself, resulting from a 30 ft. high shale slope forming the right side of the main river channel. This explanation was difficult to accept since no cracks were seen on the crest and the conditions tending to cause differential settlement were not extreme.	In spite of an excellent growth of grass which is mowed and fertilized, they have had a problem with jugs appearing each year for 20 years. They fill them with sand and sod. In Jan. 1971 there were many jugs to 2 ft. or more in diam. The grass cover could not have been thicker or more protective. (See photo 8, Appendix J). The crest is paved with asphaltic concrete and upstream slope has thick blanket of riprap in good condition.



Locations of Oklahoma Failed Dams

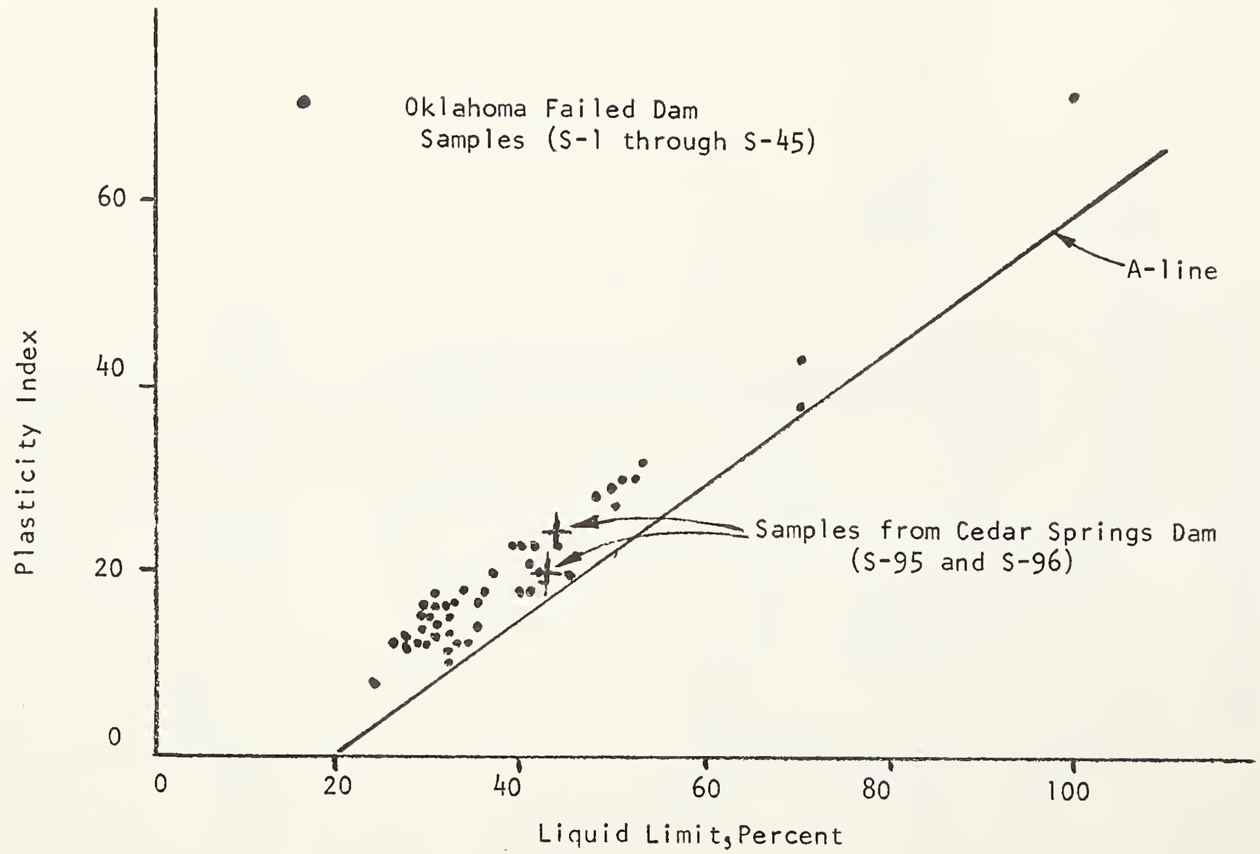
EXHIBIT 2



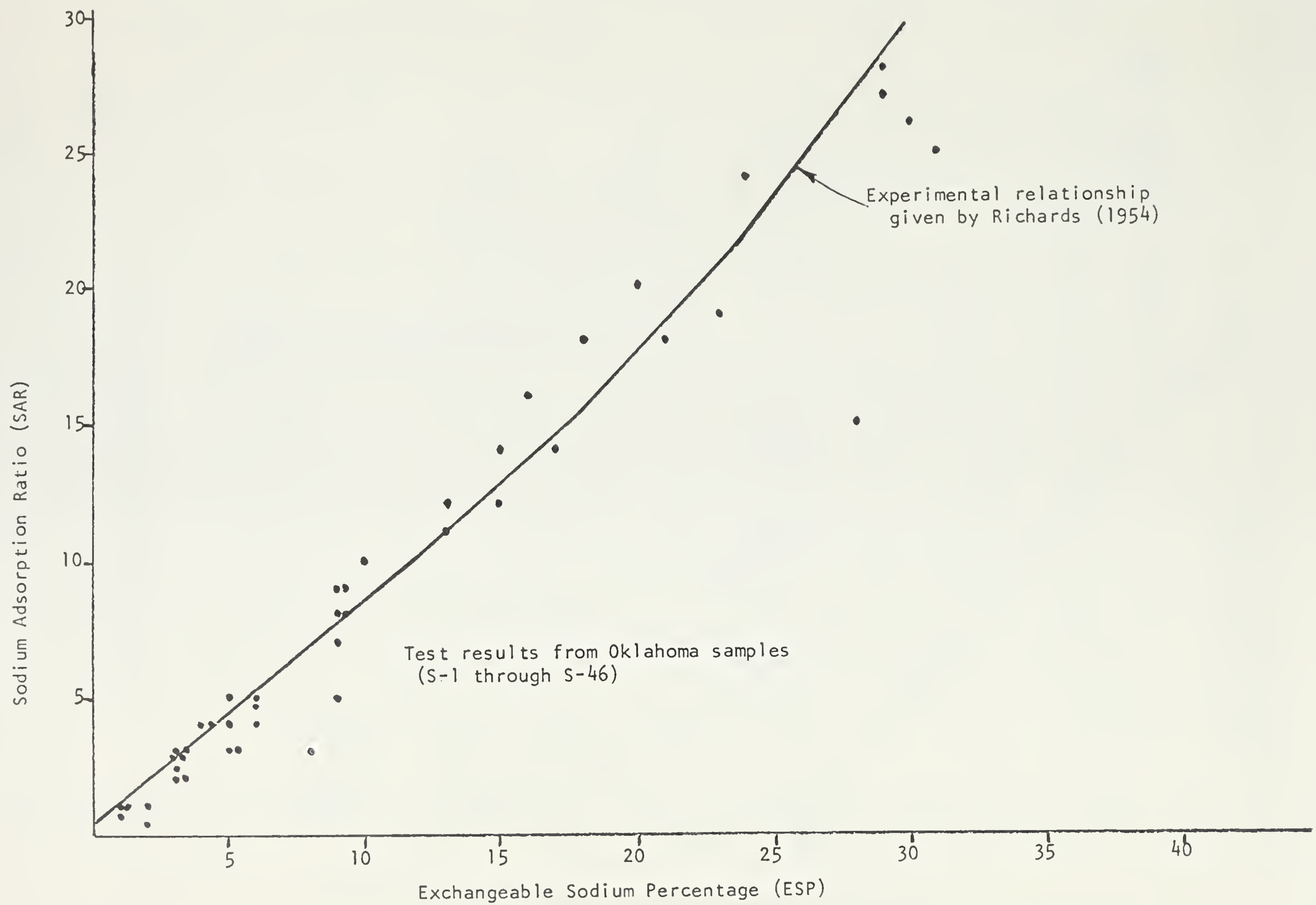
Areas of Tunnel Erosion from Rainfall in
Mississippi and Arkansas

EXHIBIT 3

EXHIBIT 4

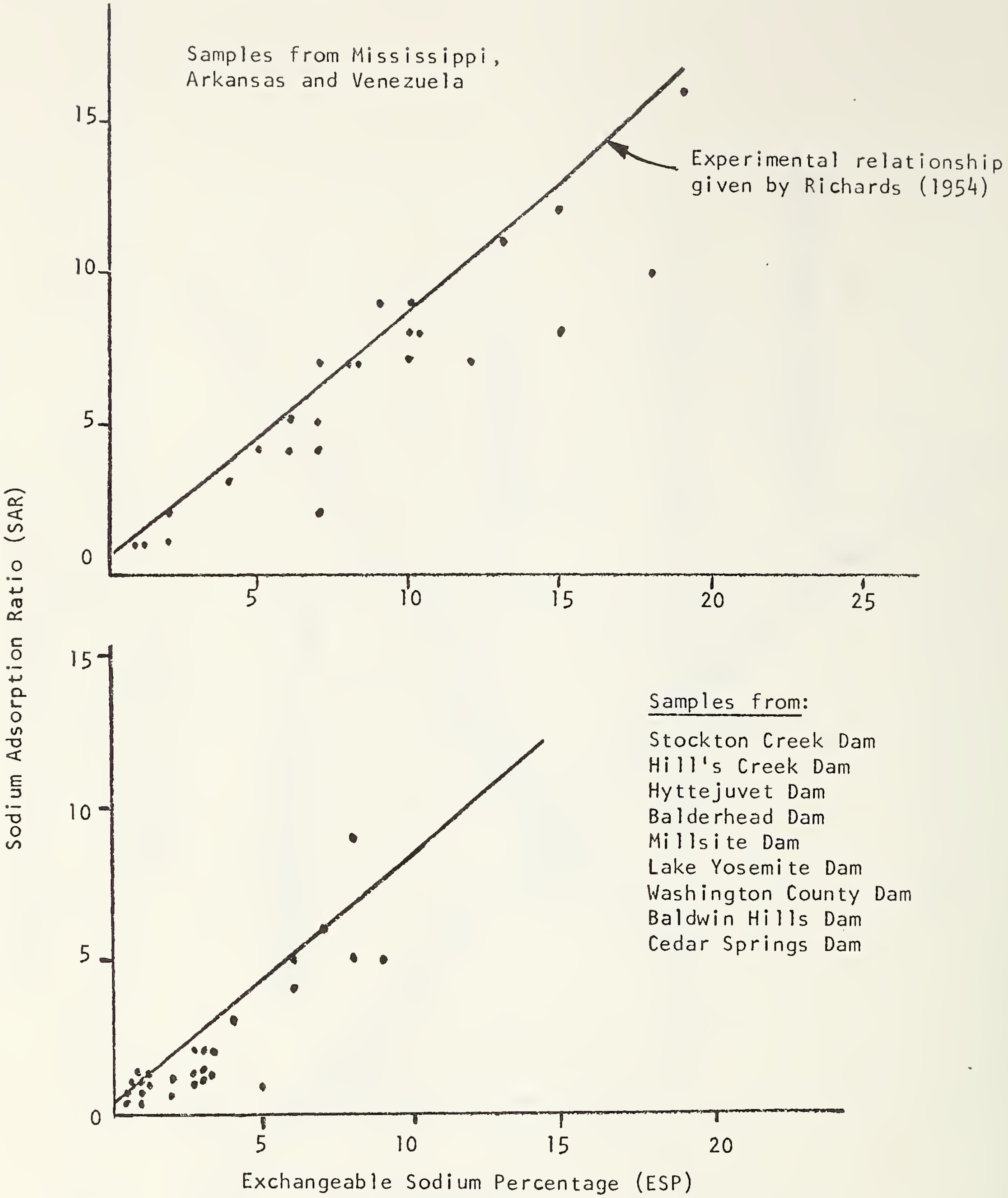


Range of Atterberg Limits for Samples From Failed Oklahoma Dams



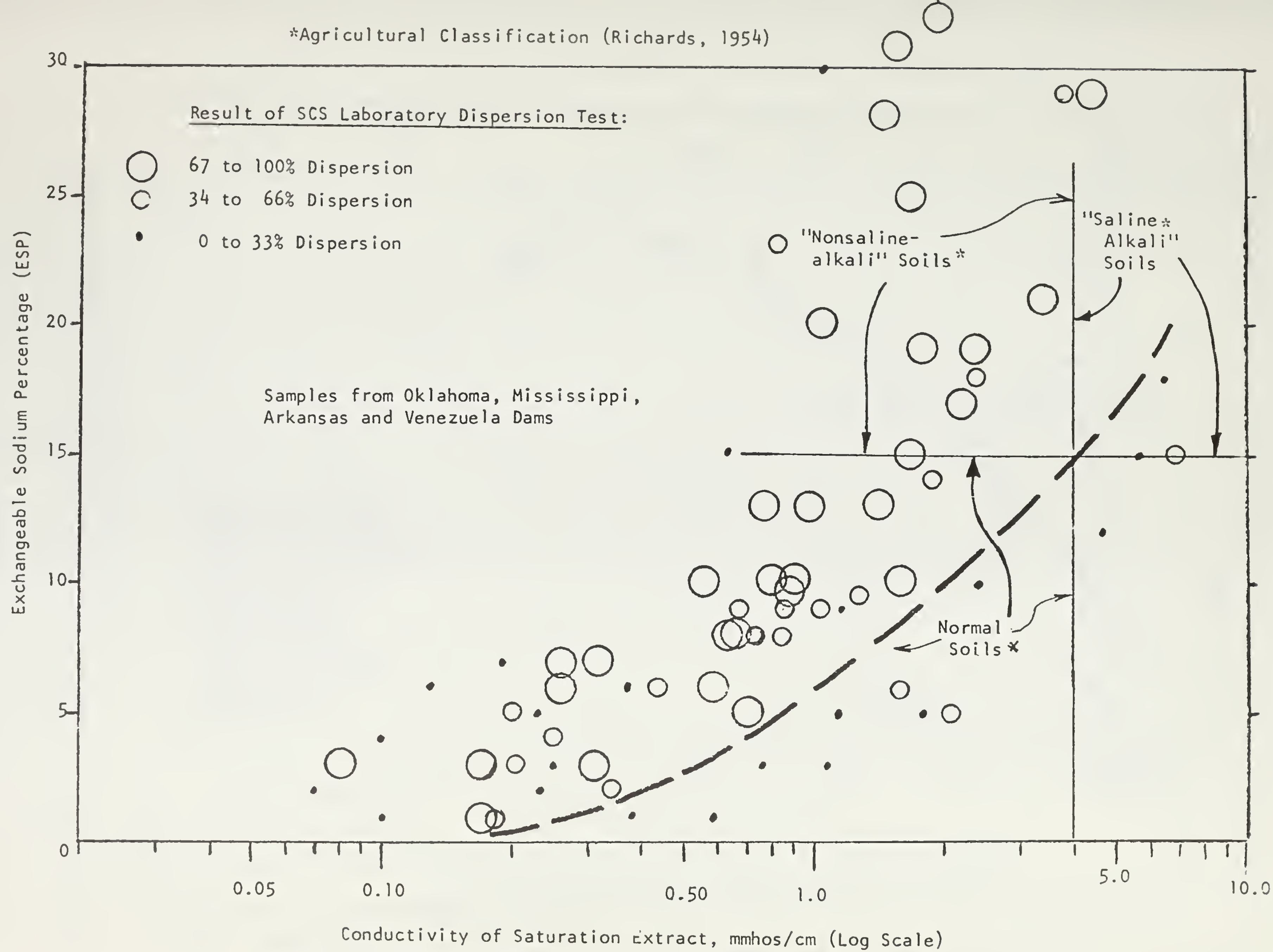
ESP and SAR Values for Samples from Oklahoma Failed Dams

EXHIBIT 5



ESP and SAR Values for Samples from
Mississippi Dams and Others

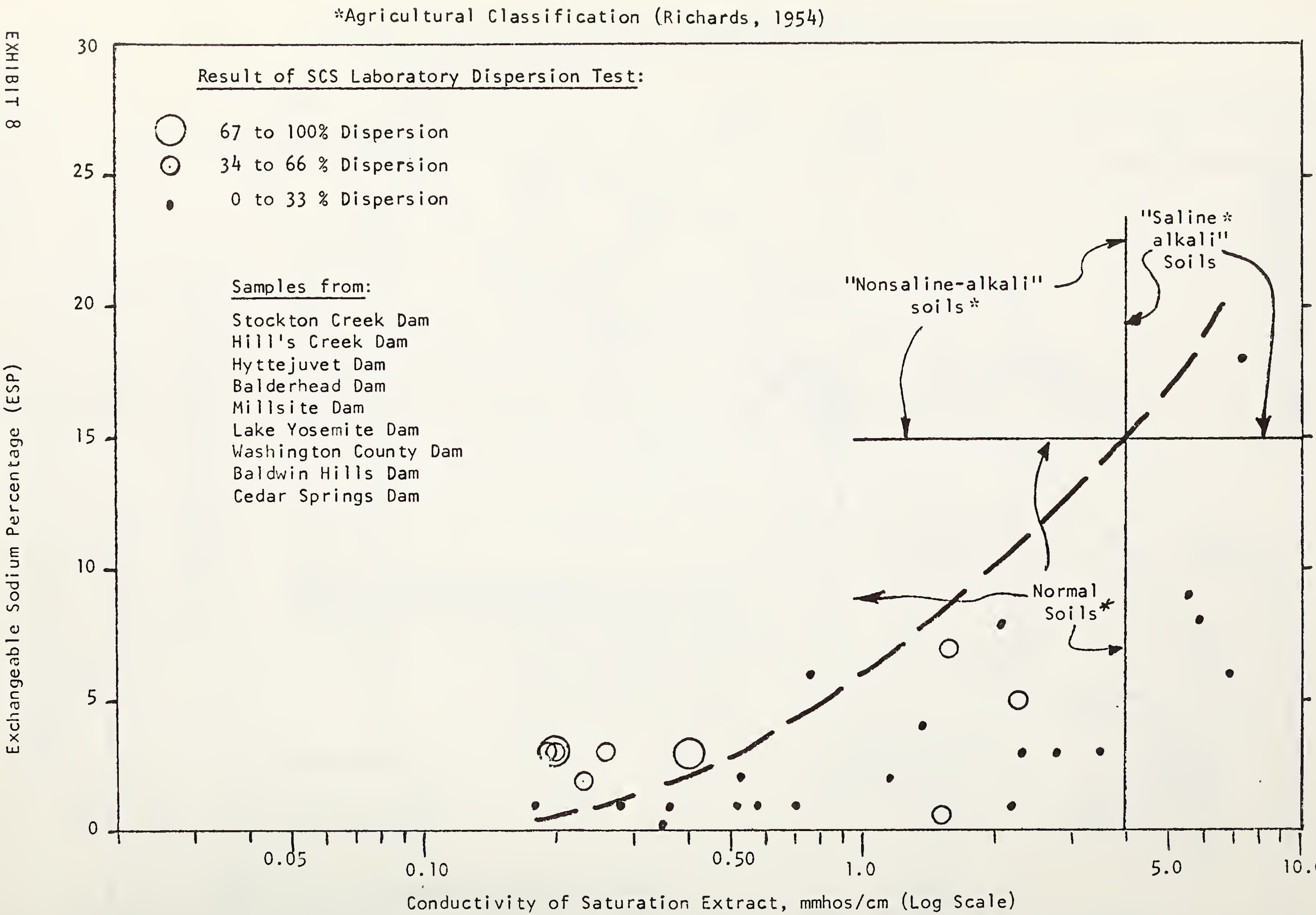
EXHIBIT 6



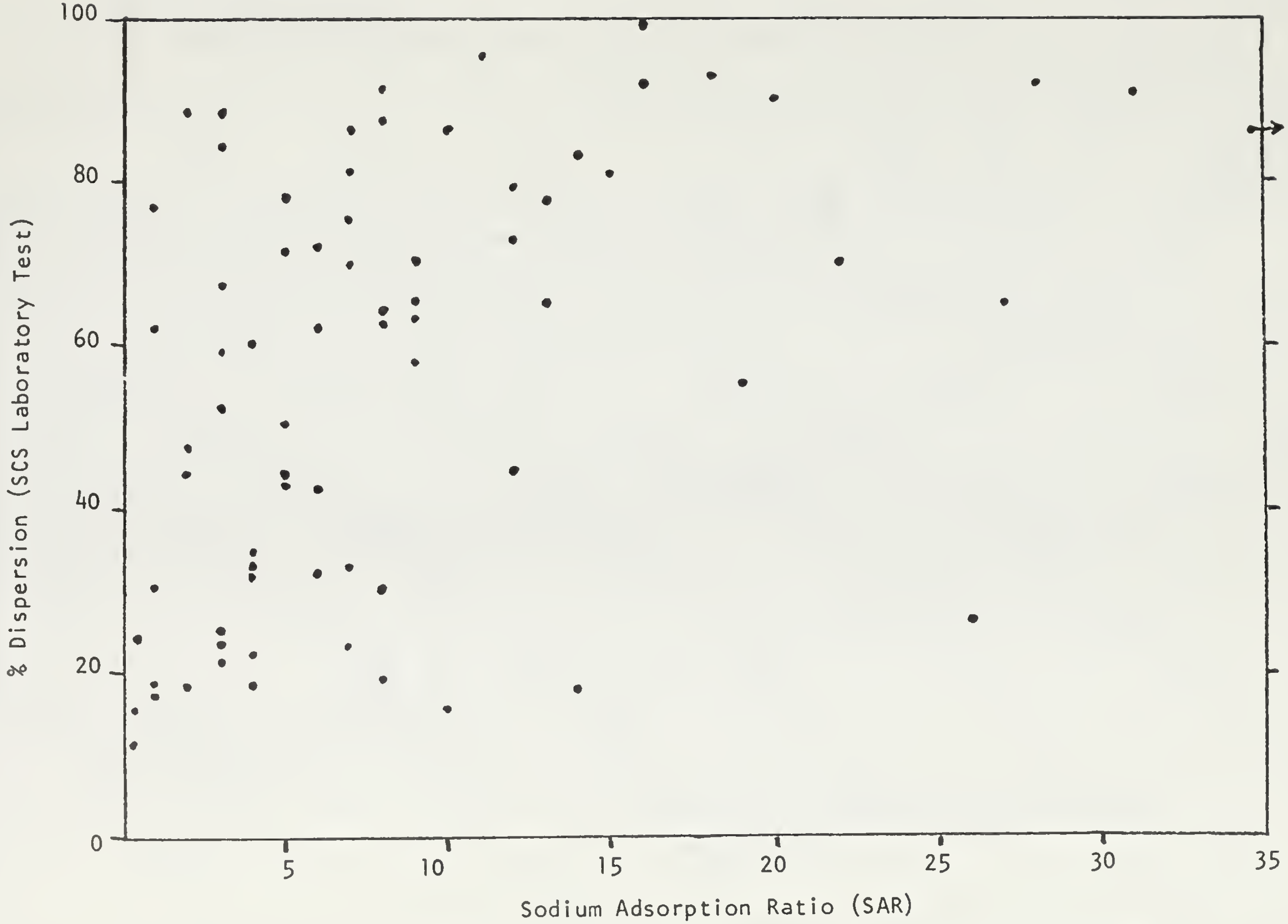
Laboratory Dispersion as Related to ESP and Conductivity of Saturation Extract

EXHIBIT 7

EXHIBIT 8



Laboratory Dispersion as Related to ESP and Conductivity of Saturation Extract

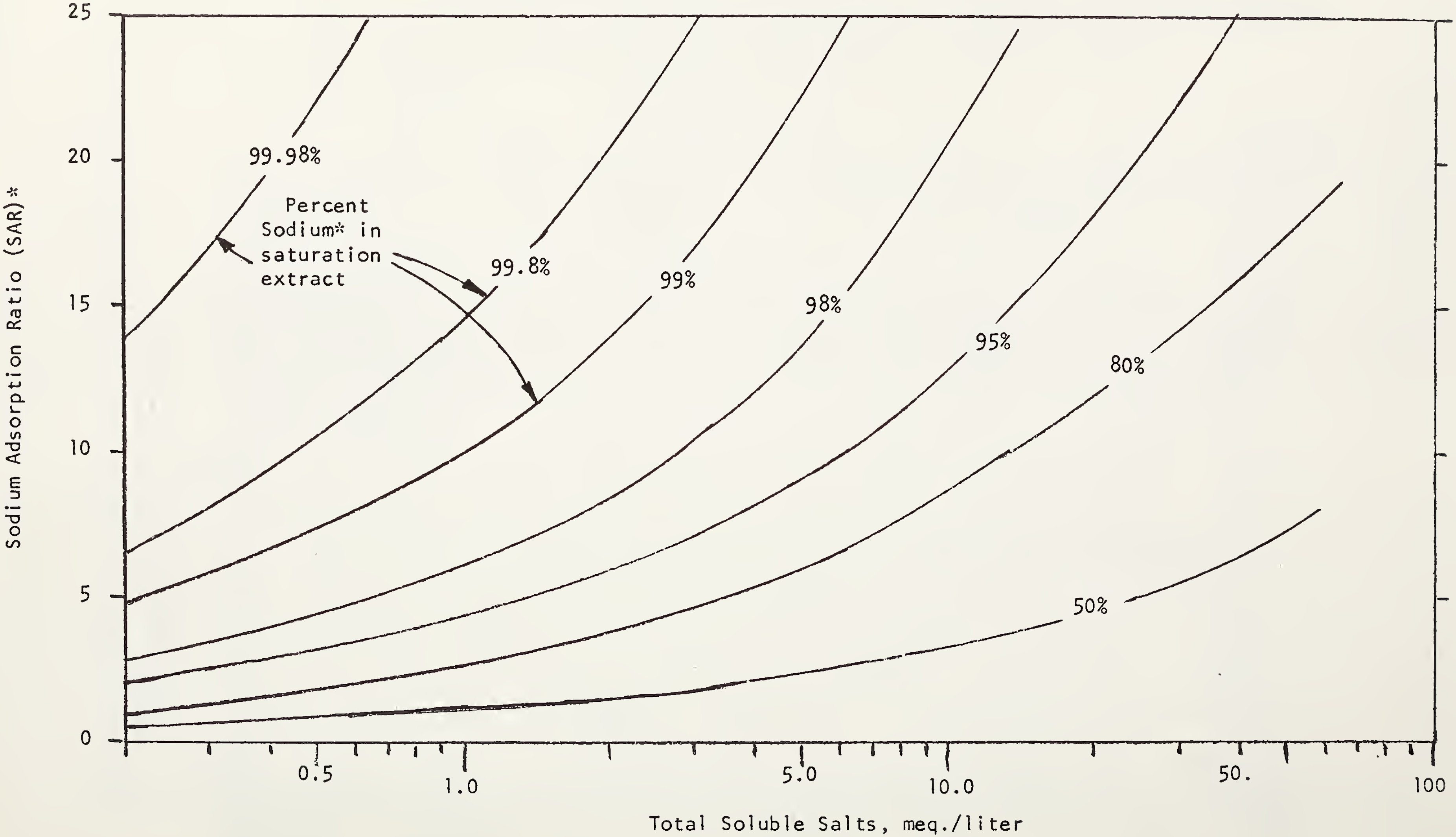


Laboratory Dispersion Test Results as Related to Sodium Adsorption Ratio Alone

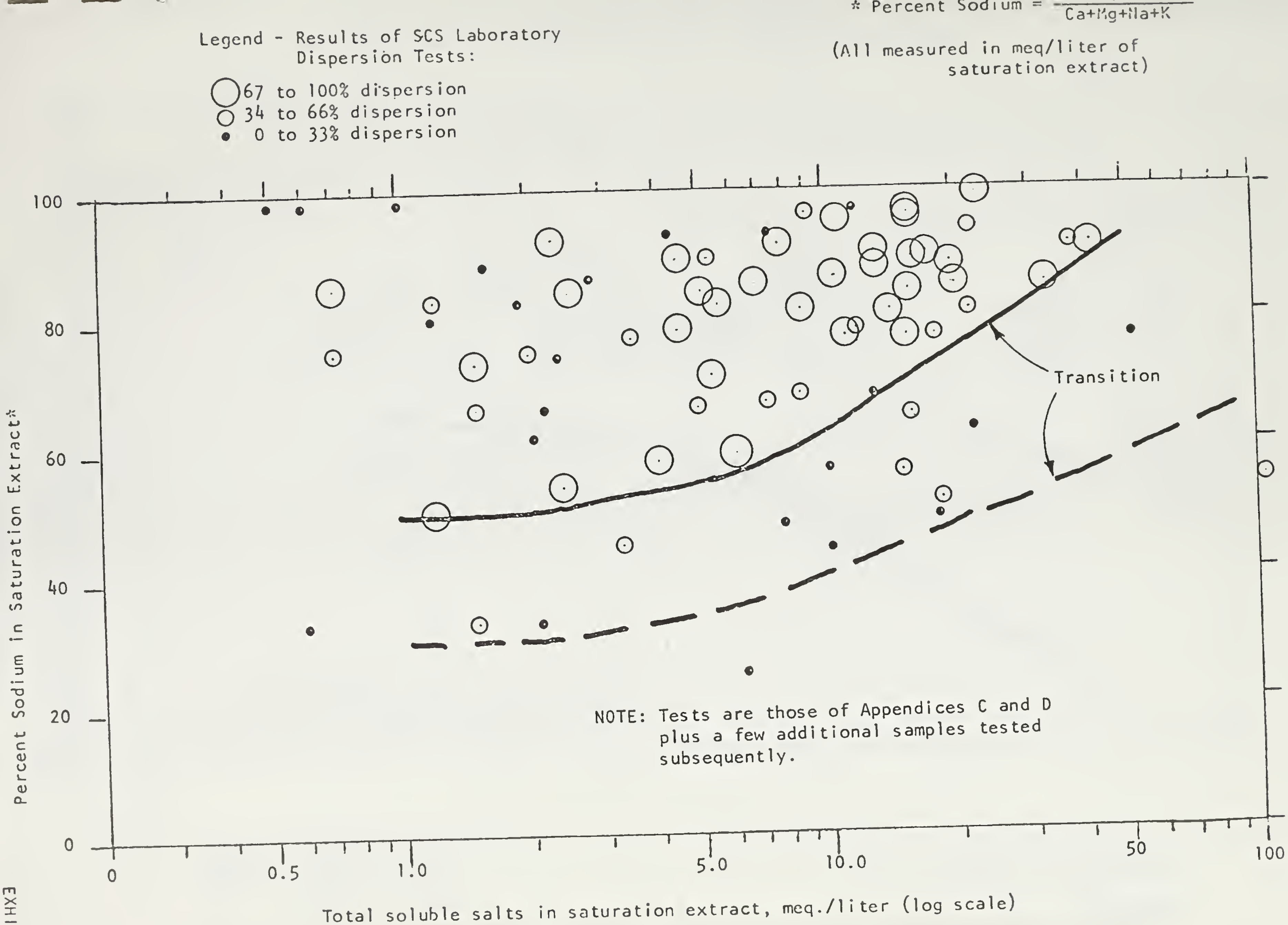
(For samples from Oklahoma, Venezuela, ARkansas and Mississippi Dams)

EXHIBIT 10

$$^{*}SAR = \frac{Na}{\sqrt{(Mg+Ca)/2}} \; ; \; \text{Percent Sodium} = \frac{Na}{Ca+Mg+Na} \text{ - all in meq./liter}$$



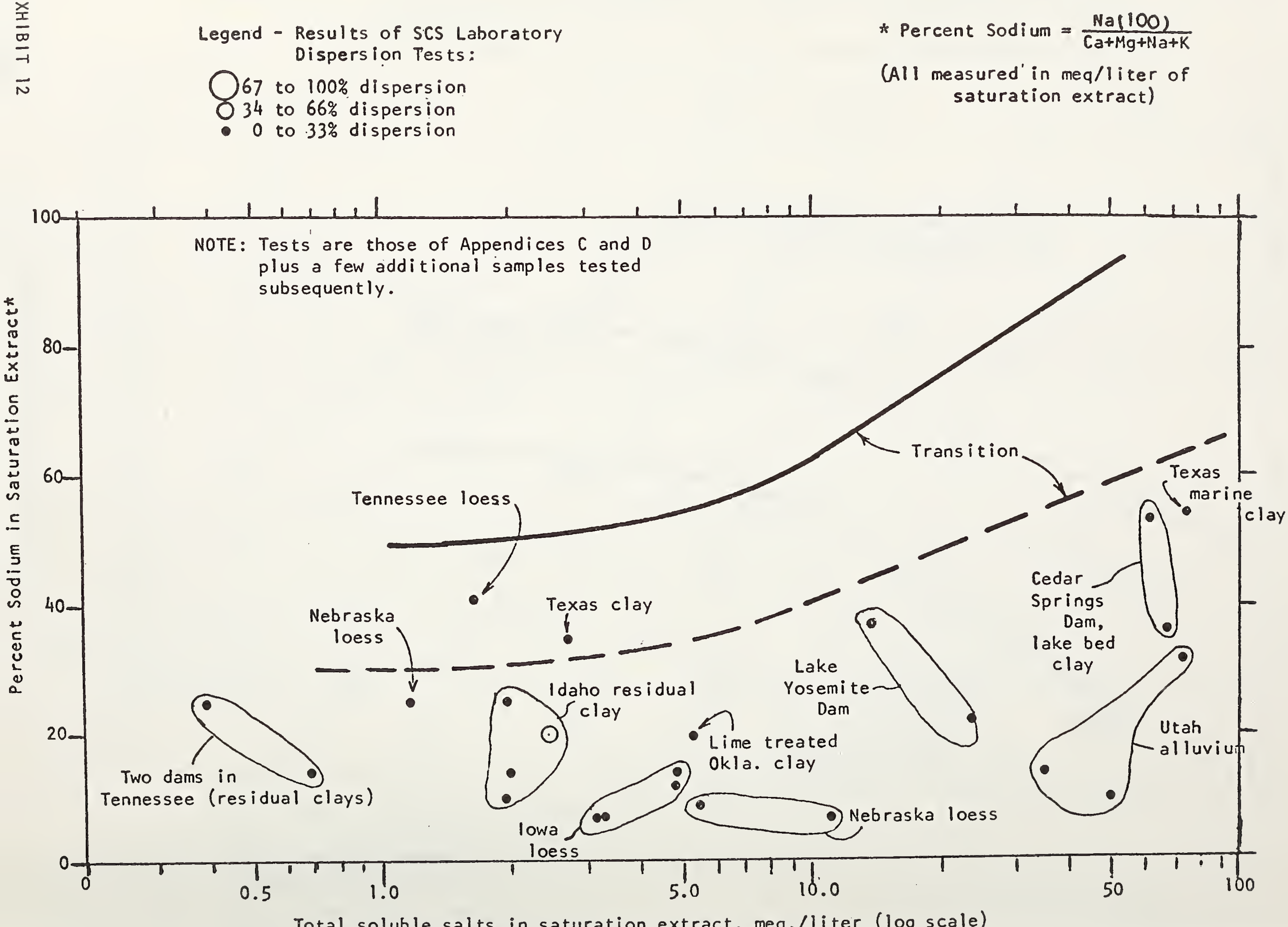
Comparison of SAR and Percent Sodium in Saturation Extract (Calculations only)



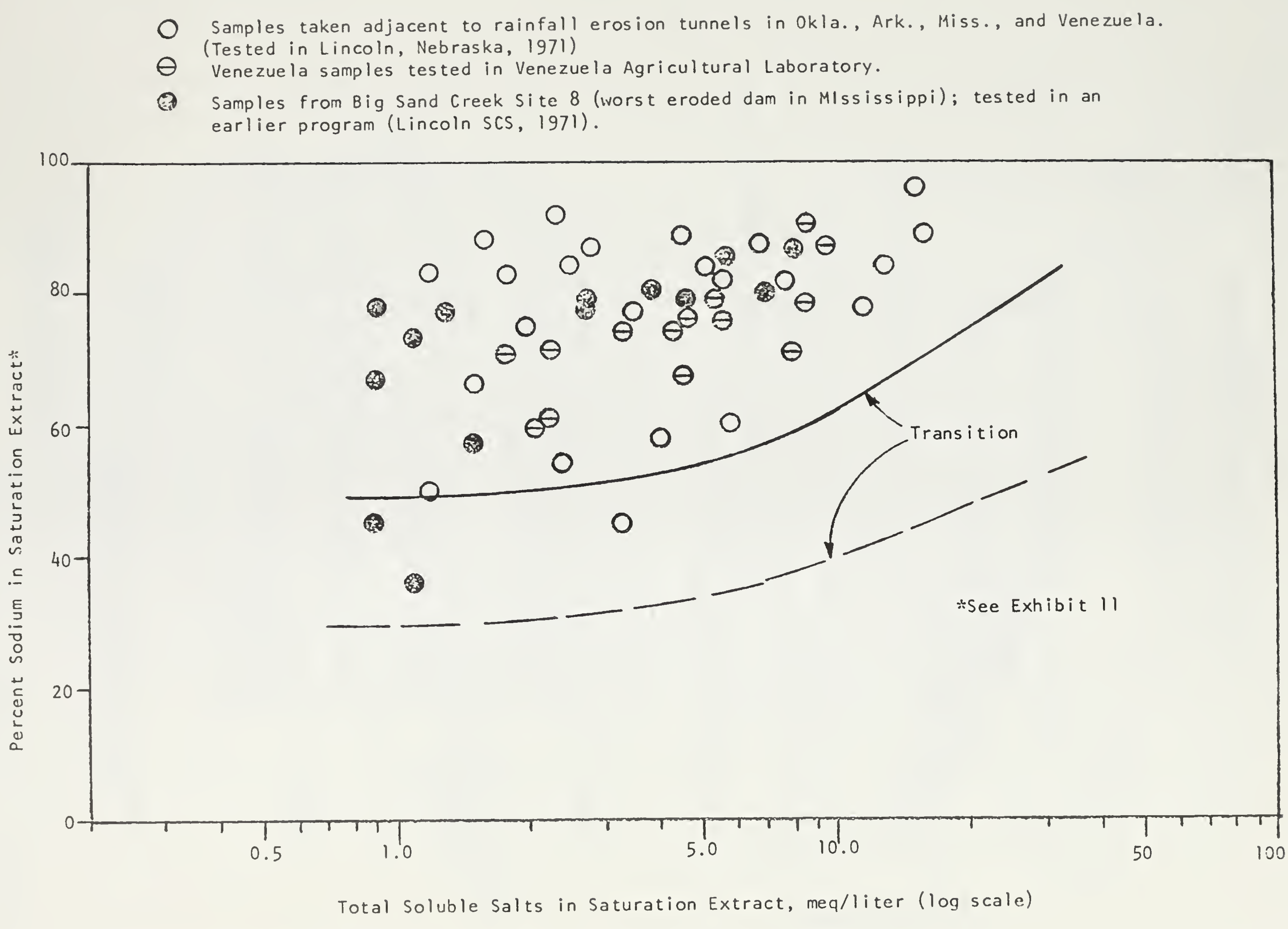
RESULTS OF TESTS ON SAMPLES FROM DAMS WHICH FAILED BY BREACHING OR WERE DAMAGED BY RAINFALL EROSION TUNNELS

EXHIBIT 11

EXHIBIT 12



RESULTS OF TESTS FROM UNDAMAGED DAMS (CONTROLS)

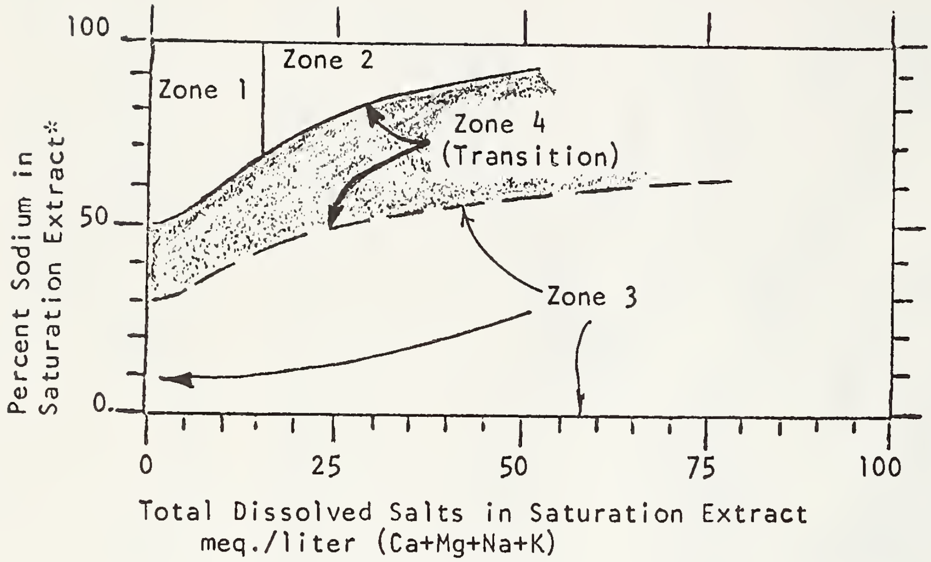


TESTS ON SAMPLES FROM EMBANKMENTS DAMAGED BY RAINFALL EROSION TUNNELS

EXHIBIT 13

$$\text{* Percent Sodium} = \frac{Na(100)}{Ca+Mg+Na+K}$$ All in meq/liter of Saturation Extract.

Note: The Zone 4 shown below is the same as the "Transition" shown in Exhibits 11 & 12 in logarithmic plots.



Zones 1 and 2 include nearly all of the clay samples from dams which failed by breaching in Oklahoma and Mississippi. Samples generally have high dispersion when tested in the laboratory. Highly erodible clays.

Zone 1 includes all samples from 16 clay dams which were damaged by tunnel erosion from rainfall in Venezuela, Oklahoma, Mississippi, Arkansas, Tennessee and Texas.

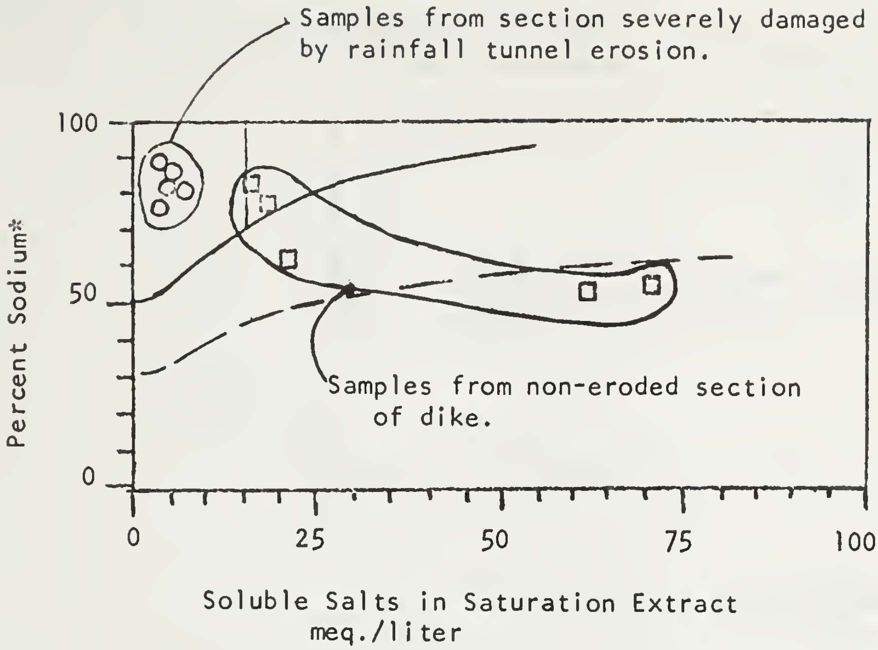
Zone 3 includes the test results for most of the "control" samples. Probable range of ordinary, erosion resistant clays.

Zone 4 is the transition zone. Most samples in this zone had low dispersion when tested in the laboratory. The lower boundary of the Zone is not well established by the data.

SUMMARY OF CORRELATION BETWEEN CHEMICAL TEST RESULTS AND DAM PERFORMANCE EXPERIENCE

EXHIBIT 14

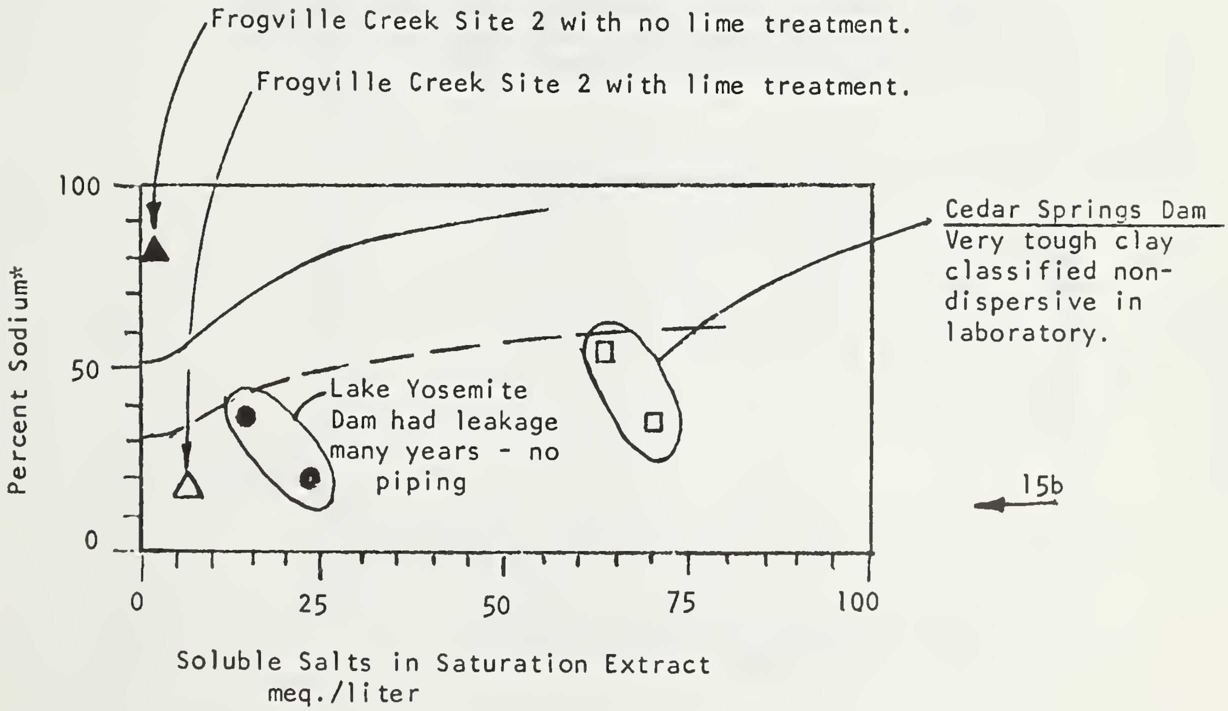
* See definition on Exhibit 11



Venezuelan Dike Samples

15a

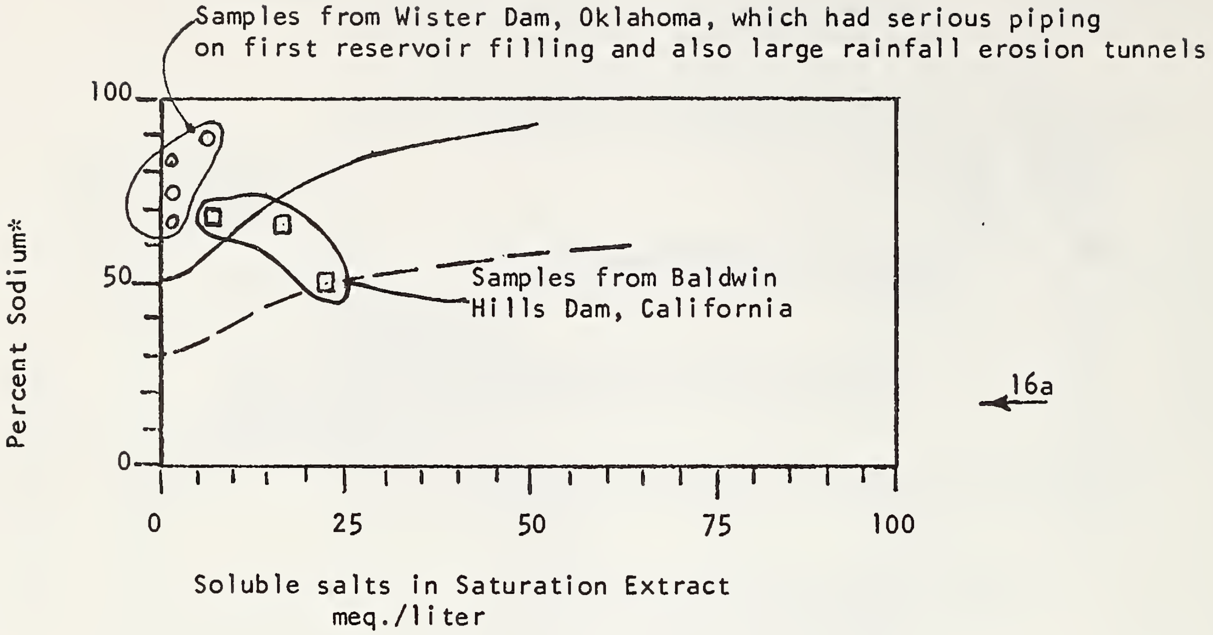
Note: See Exhibit 14 for general correlation



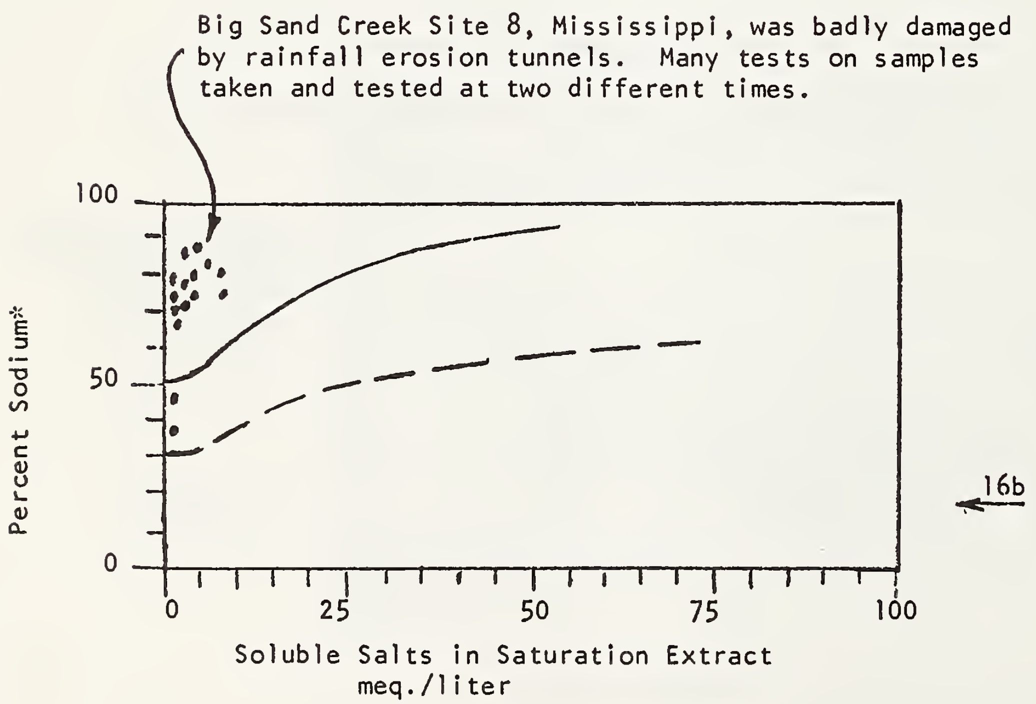
Results of Chemical Tests for Some Individual Examples

EXHIBIT 15

*See definition on Exhibit 11.



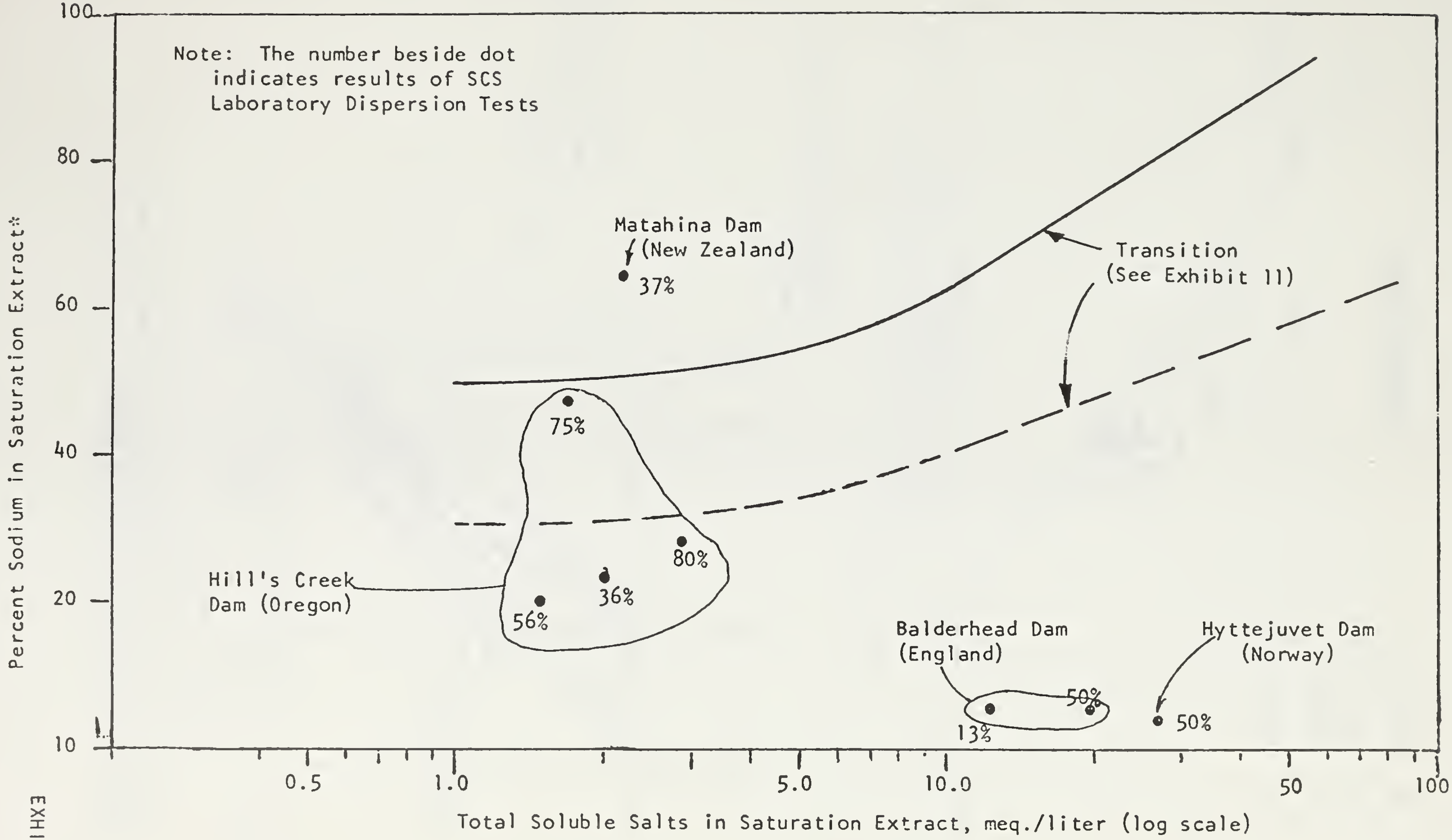
16a



16b

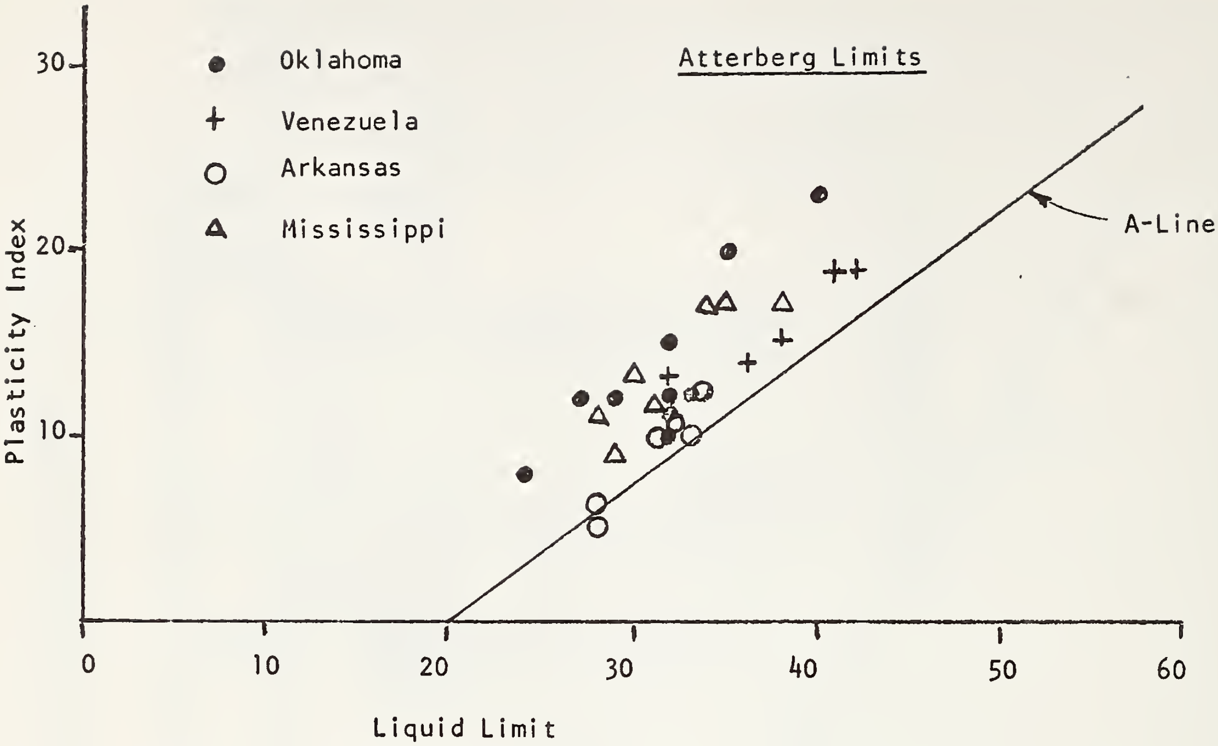
Results of Chemical Tests for Some Individual Examples

*See Exhibit 11.

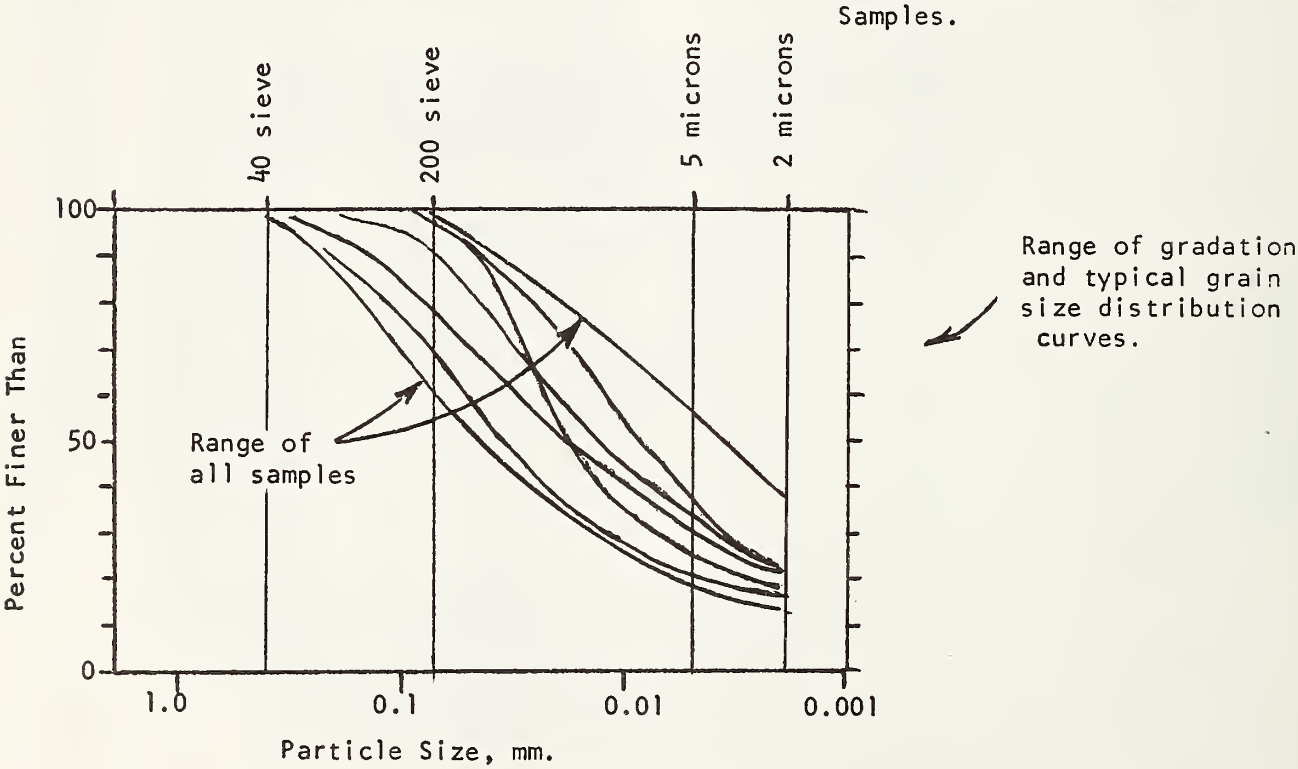


TESTS ON CORE MATERIAL OF ZONED DAMS DAMAGED BY PIPING

EXHIBIT 17



Note: See Exhibit 13 for Chemical Tests on these Samples.



Atterberg Limits and Gradation for Soil Samples Taken Near Vertical Rainfall Erosion Tunnels

APPENDIX A

PIPING AND EROSION IN CLAY
RECENT INVESTIGATIONS AND LITERATURE

PIPING AND EROSION IN CLAY
RECENT INVESTIGATIONS AND LITERATURE

Original research has been carried out in Australia in the last few years relating the incidence of piping in small clay dams (for farm ponds) with the chemical characteristics of the clay. Aitchison and Ingles (1964), Aitchison and Wood (1965), and Rallings (1966)[1] report studies of many low dams (usually less than 20 feet in height) in various parts of Australia. These are generally homogeneous embankments without filters or drains constructed without moisture control and compacted only by the travel of hauling equipment. Because of the prevailing climatic conditions the embankments were usually built dry of optimum. Of more than 3000 such structures counted, roughly 8% failed by rapid progressive erosion of spontaneous leaks. Nearly 100 of the failures were studied. Tests of the soil chemistry as well as of the engineering properties of the soil were made. The investigators concluded that there is a strong correlation between the likelihood of failure and the presence of soils with high percentages of exchangeable sodium cations, especially in the case where the reservoir water has a low content of soluble salts. Similar results have been reported from a study of ten small dams in Israel, five of which failed by piping (Kassiff and Henkin, 1967). Some of the main conclusions and observations from these studies are:

1. While it has been commonly believed by the civil engineer that clays are resistant to erosion, in fact there is great difference in erosion susceptibility. Some clays are highly susceptible to erosion and piping failure.

2. The piping of the clay is caused by a process called "dispersion" (or "deflocculation"). This occurs when the repulsive forces (electrical surface forces) between the individual clay particles exceed the attractive (Van der Waal) forces so that, when the clay mass is in contact with water, individual clay particles are progressively detached and go into a more or less permanent suspension. If the water is flowing the dispersed individual clay particles are carried away.

3. The tendency for dispersion piping is governed by a number of properties of the clay, including the exchangeable sodium percentage (ESP), the pH, the soil type and the content of dissolved salts in the water. The sodium acts to increase the thickness of the diffuse double layer and to increase the repulsive forces between particles, making it easier for individual clay particles to be detached from the mass.

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1) See References

4. The lower the content of dissolved salts in the eroding water, the greater the tendency for the clay particles to go into suspension and for erosion to occur.

5. When a concentrated leak starts through an embankment of dispersive clay, either of two actions can occur: (1) if the velocity of flow is sufficiently low, the clay surrounding the flow channels swells and progressively seals off the leak, or (2) if the initial velocity is sufficiently rapid, the dispersed clay particles are carried away, enlarging the flow channel at a faster rate than it is closed by swelling and the dam fails by piping.

6. The initial leaks may be through channels built into the clay embankment by inadequate compaction (macropores between soil chunks) or through drying cracks.

7. Soils with an ESP of 7 to 10 are moderately dispersive and were associated with piping when the reservoir had water with a low content of dissolved salts. Soils with ESP exceeding 15 have serious piping potential.

8. There is no good relationship between the piping potential and the results of ordinary engineering index tests for clays (Atterberg Limits, particle size, shrinkage). Soils with high contents of exchangeable sodium range from cohesionless silts (ML) to clays of high plasticity (CH).

9. High ESP values and piping potential are generally found in soils where the clay fraction is composed largely of montmorillonite. Some illites have high ESP values. High ESP kaolins are rare. Also, kaolins tend to cause less trouble because the dispersed particle size is much larger than montmorillonite and a larger initial leakage channel is required for piping to start.

10. A simple and rapid dispersion test ("crumb test") was found to give good correlation with the incidence of piping failure. Briefly, the test consists of placing air-dried crumbs of soil in a one milliequivalent/liter solution of sodium hydroxide. If, after a period of one hour, a colloidal cloud of dispersed clay exists around the soil crumbs, then the soil is considered to be dispersive. "The test is considered to be technically superior to the chemical analysis as it automatically accounts for the ESP and clay type. Soils which disperse in this test cannot be used safely in farm dam construction without compaction and moisture content control." (Rallings, 1966) In 25 of 26 dams which failed, Rallings found that the soil showed dispersive characteristics in this test.

11. "One method of piping control often proposed is to place an inverted filter at points of seepage outflow. However, filters built to conventional rules cannot prevent the passage of a suspension of deflocculated clay" because the individual clay particles are too small. "In the authors' opinion, therefore, no reliance should be placed on filters in these circumstances (dispersive type piping)". (Aitchison and Wood, 1965)

The studies described above were based on the performance of small farm type dams, which were generally constructed with tractors and scrapers with no roller and without engineering control. In their early papers the Australian investigators speculated that piping in clay cores had not occurred in well-constructed dams, even when the soil had a high ESP and the reservoir water was low in dissolved salts.

Recently, however, some Australian engineers have concluded that piping of highly dispersive clay cores may have developed even in dams constructed with rollers and moisture-density control. Ingles, et al (1968 and 1969) describe the failure of Flagstaff Gully Dam, retaining a small municipal water supply reservoir (50 million gallons) for the city of Hobart, Tasmania. The dam, 55 feet high and 690 feet long, is a rock-fill dam founded on rock with a central clay core, comprised primarily of montmorillonite, with upstream and downstream filters. The clay comprising the lower half of the core had the following average properties:

Liquid Limit = 40
Plasticity Index = 25

Particle Size Distribution:

Size	Average Percent Finer
2 mm	90%
200 sieve	65%
0.002 mm	30%

The upper elevations of the clay core, taken from another borrow pit, had a liquid limit of 75 and a plasticity index of 60. ESP in the lower portion of the core was 17 and the upper portion 10. The core was apparently well constructed and at a water content near Standard AASHO optimum.

The reservoir was filled with water having a low dissolved salt content from a source 20 miles away. (No local runoff was permitted to enter.) The total of all dissolved salts in the water was about 1.0 milliequivalents/liter. In July 1963, three weeks after the first filling, a large dirty leak appeared at the downstream toe. Nearly all of the water in the small reservoir (about 40 million gallons) was lost. Several samples of the dirty leakage water were tested; the content of suspended clay particles was about 1000 ppm and the average size of suspended particle was about 0.6 microns. In subsequent explorations it was found that there were two erosion tunnels through the core just above the bedrock surface with diameter of about 2 to 3 feet. As in most such cases, the cause of the failure was not determined reliably; however, because of the coincidence of the high exchangeable sodium and the very pure reservoir water, the investigators concluded that dispersion of the clay could have been a contributing cause.

James and Wickham (1970) describe the failure of another small dam (Morwell Dam) in 1968 in Australia. It was a homogeneous dam of clay of

medium plasticity (CL-CH) and a height of about 30 feet, constructed according to modern practice with moisture-density control. The average construction water content was about 2% below Standard AASHO optimum and the average density was about 98% of Standard AASHO maximum dry density. When the reservoir filled rapidly for the first time in December 1968, a leak of about 100 gpm broke out at the downstream toe through an erosion pipe about one foot in diameter in the clay embankment (See Photo No. 24, Appendix J). The small reservoir was rapidly lowered to prevent more extensive damage.

When the embankment was only a few feet high, work was stopped for three months. During this period the weather was very dry. The leaks and erosion pipes subsequently developed at about this level. The rapid development of piping in a well constructed and impervious clay dam cannot be easily explained unless the material were highly susceptible to piping. Investigations showed that the clay did not have very high ESP (5 to 7), but it did nevertheless have some reaction to the crumb test. The reservoir water was found to have a total cation concentration of about 5 milliequivalents/liter.

As part of the remedial measures, gypsum (calcium sulphate) was spread on the upstream slope of the dam, in a layer about one inch thick (about 30 tons). "As the reservoir water is used and replenished, and the salt concentration falls, the gypsum blanket will still be relied upon to increase the concentration of divalent salts in the seepage water entering the embankment (and prevent dispersion) until the clay swelling closes off any pervious channels."

The Australian investigations have been carried out by and under the sponsorship of the CSIRO, the Water Research Foundation of Australia and several universities. Several continuing studies are currently in progress. Ingles (1968) and Aitchison and Ingles (1969) contain recent summaries of the present understanding of the fundamental theoretical relationships between clay chemistry and erodibility.

There has been very little research in the last few years in the United States. Parker and Jennie (1967) and Bell (1968) describe studies of severe erosion of clay from rain runoff to highway embankments and bridge abutments in western United States. Their general conclusions are that severe erosion occurs generally in silty and clayey materials that contain a high percentage of exchangeable sodium and that contain at least 20% montmorillonite.

While this is a new subject for the civil engineer, it is well known by agronomists that clayey soils with ESP of 15, or more, are poor for agriculture because they are "dispersive" (Richards 1954). For such soils the ground surface frequently becomes highly impermeable, probably because of plugging of the soil pores by dispersed clay particles washed downward by runoff.

As early as the mid-1930's the Soil Conservation Service had already recognized that certain clayey soils in localized regions in the western United States were particularly susceptible to erosion and a few descriptions were published of tunnel erosion damaging embankments, canals and causing subsidence craters and sinkholes in cultivated fields (Carroll, 1949). Also, the SCS developed a direct laboratory test for measuring clay dispersibility (Volk, 1937), which test is still employed to identify highly erodible soils (see test procedures in Appendix H, pages 1-1 and 1-2).

APPENDIX B

DESCRIPTION OF DAMS FROM WHICH
SOIL SAMPLES WERE TAKEN AND TESTED

DESCRIPTION OF DAMS FROM WHICH SOIL SAMPLES WERE TAKEN

Soil Sample Number	Dam Name and Brief Performance Description
(See Appendices C & D)	
S-1 through S-41	Soil Conservation Service Dams in Oklahoma damaged by piping. Detailed descriptions given in Exhibit 1.
S-42 through S-45	Wister Dam is a homogeneous clay embankment in southeastern Oklahoma. On the first reservoir filling in 1949, a large dirty concentrated leak developed through the dam under low-head (See Exhibit 1). The trouble was very similar to the piping failures which occurred in the 11 SCS clay dams in Oklahoma, except that the dam and reservoir were much larger. After repairs (grouting, sheet piling, and berms), there has been no further leakage or piping. Erosion tunnels have developed from rainfall on the downstream slope (see Photo 8, Appendix J), very similar to those experienced on the SCS dams in Oklahoma and Mississippi, in spite of the fact that the slope is covered with an excellent protective growth of grass which is fertilized and mowed at regular intervals. Many new tunnels develop on the downstream slope each year. These are generally repaired by filling them with sand. Samples of the typical embankment clay were taken from the walls of four of these vertical erosion tunnels at widely spaced intervals by courtesy of the Tulsa District, Army Corps of Engineers. See Bertram, 1967, and U. S. Army Engineers, 1959, (Bibliography) for details of design, construction and performance.
S-46 through S-57	Flood control dike on the east bank of the Rio Zulia in western Venezuela. Dike was built of clay using moisture-density control and a sheepsfoot roller in 6-inch layers. Soon after completion of construction in 1963-64, certain lengths of the dike were badly damaged by tunnel piping caused by rainfall. The nature of the erosion and piping tunnels is very similar to that which damaged Soil Conservation Service dams in Oklahoma and Mississippi (see Photos 1 and 2, Appendix J). Samples were taken from the damaged and undamaged sections of the dike and were shipped to the United States for inclusion in this study by courtesy of the Venezuelan Ministry of Public Works, Hydraulic Works Department, Sr. R. Martinez-Monro, Director.
S-58 through S-60	Stockton Creek Dam, California. This is a well constructed 80-foot high homogeneous dam which failed on the first reservoir filling in 1950 by piping. The failure was similar to the SCS Oklahoma piping failures in many main details (Exhibit 1):

Soil Sample Number	Dam Name and Brief Performance Description
S-58/60 cont'd	1) No obvious cause of the failure was found even after a thorough post-failure study; 2) The dam was breached just at the contact with the rock abutment, where the tendency for differential settlement crac-ing was greatest, but no cracking was seen; 3) The failure occurred without witnesses on the first rapid reservoir filling; 4) No differential settlement cracks were seen; 5) No loss of life or appreciable property damage was threatened; 6) After repair, the dam has performed satisfactorily. Some details of the failure are given in Terzaghi and Peck (1967) p. 596-7, and in Sherard (1953). Soil samples for testing in this study were taken by J. L. Sherard. The soil is a residual weathered-in-place product of the disintegration of a sound and massive schist.
S-62 through S-64	Hill's Creek Dam, Oregon, is a 300-foot high earth dam with a central core and gravel shells (see cross-section in Sherard, Woodward, Gizienski and Clevenger, 1963, p. 47). The core consists of clayey gravel, the clay fraction being comprised largely of halloysite. Soon after the first reservoir filling in 1963, a wet strip developed on the downstream slope near the left abutment over a length of about 500 feet and at an elevation of about 50 feet below the full reservoir level. This wet spot appeared each year during full reservoir, but appeared to be decreasing, until in July 1969 the leak abruptly increased to about 0.8 cfs. The core was grouted and no leakage developed when the reservoir was filled in 1971. The cause of the initial leak and abrupt increase in flow is not well known. The soil samples were obtained from large diameter auger holes put down in the core to study the problem in 1970, and were obtained for this study through the courtesy of Mr. Robert J. Pope, Materials and Foundations Branch, Portland,District, Army Corps of Engineers. In addition to the leakage problem, it was of interest to include samples of halloysitic clay to compare with the test results from the more common clay minerals.
S-65	Lost Creek Dam, in Oregon, is not yet constructed. A typical sample of a prospective borrow material comprising halloysitic clay minerals was included in the study to compare with the tests on the Hill's CReek samples described above, courtesy Mr. R. J. Pope.
S-67	Hyttejuvet Dam in Norway. This is a rockfill dam with thin, vertical central earth core (glacial moraine in form of a gravelly, silty sand), about 300 feet high, completed in 1965. On the first reservoir filling in 1966, a dirty leak of about 60 liters/sec. appeared abruptly at the downstream toe when the reservoir was nearly full. Subsequent studies led to the conclusion that the water broke through a crack in

Soil Sample Numer	Dam Name and Brief Performance Description
S-67 cont'd	the core which was caused by differential settlement between the core and shells. It was believed that the crack was caused to open by the pressure of the water (hydraulic fracturing). The core was subsequently grouted. The experience is well described in Kjaernsli and Torblaa (1968). The sample of typical core material included in this study was provided by courtesy of Mr. Bjorn Kjaernsli, Norwegian Geotechnical Institute.
S-68.1 and S-68.2	The Balderhead Dam, in England, is a rockfill dam with thin, vertical central core (glacial moraine in form of gravelly, clayey sand), about 140 feet high, completed in 1965. On the first reservoir filling in 1966 a leak of about 35 liters/sec. appeared abruptly when the reservoir was nearly full, later increasing to a maximum of about 60 liters/sec. Very thorough studies led investigators to the conclusion that the water broke through a crack in the core which was caused to open by the pressure of the water acting on the upstream face at an elevation where the stress acting in the core had been reduced by differential settlement (hydraulic fracturing). The dam core was badly damaged by piping. Repairs were made by grouting and a slurry wall. The experience is described in detail in Vaughan et al (1970). The samples of the core material included in this study were furnished by courtesy of Dr. P. R. Vaughan, Dept. of Civil Engineering, Imperial College, London.
S-69 through S-71	Millsite Dam, Utah, is 120-foot high earth fill dam of the SCS completed in 1971. Samples of the core material (well graded, silty, sandy gravel) were included in this study as a "control", representing a non-cohesive, silty soil, to see how the test results compared with erosion susceptible Oklahoma and Mississippi clays.
S-72 and S-73	Lake Yosemite Dam, California, is a homogeneous dam of gravelly clay, maximum height about 55 feet, 5000 feet long, retaining a fairly important irrigation reservoir. Constructed in 1884, for more than 50 years the downstream slope of the dam was wet with small concentrated leaks passing through the embankment, with no sign of piping developing. Two samples were taken by J. L. Sherard from the downstream slope in general area of the leakage, for the purpose of providing a "control" and comparing the test results with the erosion susceptible Oklahoma and Mississippi clays.
S-74 and S-75	Frogville Site 2 Dam. This is a typical, low, flood control, clay dam of the SCS in southeastern Oklahoma. During construction the embankment was seriously eroded by rainfall and it was believed that the clay was uncommonly susceptible to erosion, even when compared to the ordinary erodible Oklahoma clays. After study, the contract was amended during construction and the outer 12 in. of slopes and crest were covered with lime treated soil, using techniques previously employed at 2 other Oklahoma dams.

Soil Sample Number	Dam Name and Brief Performance Description
S-74/75 cont'd	The treated soil was mixed with lime in the borrow area and spread on the slopes and crest in two 6-in. layers. The lower layer had 3% lime and the outer layer 2% lime. Sample S-74 is a composite of the lime treated dam surface. After 2 years the dam is in good condition with negligible erosion.
S-76.1 through S-79	Samples from 3 low homogeneous clay dams in Arkansas which have moderate damage by vertical tunnel erosion from rainfall. These 3 dams are located within a small geographic area near Wynn, Ark., in a relatively localized geologic feature called the Crowley Ridge, comprised of fine grained alluvial soils with capping of loess. This is the only area in Arkansas where the SCS has experienced tunnel erosion by rainfall.
S-80 through S-84	Washington County Dam, is a homogeneous, well constructed clay dam, 30 ft. high, which failed by piping on the first reservoir filling. The experience was similar to the failures of the SCS Oklahoma Dams in many of the main details: (1) no obvious cause was found even after careful post-failure study; (2) the dam was breached just at the contact with the left abutment where the tendency for differential settlement cracking was greatest; (3) there was no loss of life or appreciable property damage; (4) no differential settlement cracks were seen; and (5) after repairs the dam has performed satisfactorily. The reservoir is located in eastern Illinois (near Nashville) in an area where there is a surface layer of loess underlain by a sandy gravelly clay (glacial drift). The loessial clay in the area is known to be sodium rich (Wilding et al, 1963) and there have been troubles with erosion from rain on canals and highways. However, it was the intention to build the dam from the underlying glacial drift. After study of the failure, the investigators concluded that the most probable cause was piping through a differential settlement crack resulting from the fact that the foundation on the left side of the breach was relatively incompressible whereas on the right side there was about 30 feet of compressible alluvium in the foundation. Details of the experience are given by Hanson (1969). Samples taken by Sherard through courtesy of Illinois Dept. of Conservation.
S-85 through S-89	Three typical SCS flood control dams (Perry Creek Site #3 and Big Sand Sites 8 and 10) in general region where most of dams are damaged by tunnel erosion from rainfall. The tunnel erosion in this area is more severe than in Oklahoma: a greater number of dams are affected, there are more tunnels on each dam and the tunnels are larger in diameter. The worst example of tunnel erosion from rainfall I saw in Oklahoma (Leader Middle Clear Boggy Site 29) was comparable to the average of the dams visited in Mississippi. Photos 3 through 6 show typical erosion tunnels in Mississippi Dams. As in many cases, the erosion tunnels occur in spite of an excellent growth of protective grass.

Soil Sample Number	Dam and Brief Performance Description
S-90 and S-91	Potacocowa Site #3. Typical SCS homogeneous clay flood control dam in area of erodible soils. Has frequent, large diameter (a man can climb into them and barely have his head stick out) erosion tunnels from rainfall. Also, the dam failed before completion of construction on August 25, 1962 when a rain caused water to back up in the reservoir to a height of only about 4 feet above the invert of the spillway conduit at the upstream end. As shown in Photos 21 and 22, the leak and piping created tunnels above and on both sides of the conduit. The subsequent investigations did not provide conclusive evidence or strong opinions concerning the failure cause. Inadequate control of conduit backfill was suspected. Of 24 field density tests all but two showed relatively high water content. Mr. H. L. Cappleman concluded, "There is no evidence that the contractor failed to properly place and compact the fill"; "The exact proximate cause of the failure cannot be determined", and that differential settlement cracking was a reasonable possible cause. The failure details were very similar to those at several of the Oklahoma sites (Exhibit 1).
S-92 through S-94	Baldwin Hills Reservoir, Los Angeles, California failed by piping and breaching in December 1963 causing loss of life and extensive property damage. The piping occurred through a compacted clay lining and the natural abutment and foundation which was comprised partially of clay of low plasticity. The events are well described by Jansen et al (1967). Samples were taken by J. L. Sherard through courtesy of the Los Angeles Dept. of WAter and Power.
S-95 and S-96	Cedar Springs Dam completed in California in 1971, is a major dam at a site where an "active" earthquake fault was recognized in the foundation during the design stage. As a part of a very conservative design, an especially tough appearing clay was located, at a haul distance of about 20 miles, which was believed to be especially resistant to erosion. Samples of the typical material were included in this study as a "control", through courtesy of Mr. Donald H. Babbitt, Head, Dam Design, California Dept. of WAter Resources, SAcramento, California. A description of the dam design and construction is given in USCOLD, 1971.
S-97	Matahina Dam in New Zealand is a rockfill dam with central earth core, about 200 feet high, completed in 1966. On the first reservoir filling in January 1967, a large, dirty concentrated leak developed. Investigators concluded that the most likely cause was a differential settlement crack. Details of the construction and leakage problem are given by Galloway (1967). The sample of typical core material was supplied by courtesy of Mr. W. M. Duncan, Chief Designing Engineer (Power), Ministry of Works, Wellington, N.Z.

Soil Sample Number	Dam and Brief Performance Description
S-98 and S-99	Braunig Lake Dam, constructed in 1963, a homogeneous clay dam with maximum height of about 80 feet, was damaged by a shear slide in stiff foundation clay in December 1969. Investigations showed that the stiff clay had deteriorated, as the result of peculiar piping in a 12-inch thick fine silty sand layer directly underneath, and must have had a final shear strength of about C=0; ϕ=9°. The experience could not be well explained. Samples were included in this study to compare with the high sodium clays of Oklahoma, courtesy of the San Antonio City Public Service Board and Mr. Ralph F. Reuss, National Soil Services, Inc., Houston, Texas.

APPENDIX C

SOIL MECHANICS LABORATORY TEST RESULTS
(See Appendix B for descriptions of dams from which samples were taken.)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA
Sheet 1 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART			MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT — FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD) MAX. γ_d p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-1	1	Okla. Little Wewoka Cr. #17.	E	41	7.5YR	5/8	1/	28	33	48	71	81	90	99	99	99	100			3	35	15	20	12.2	CL	42.5	11	111.5	16.5	2.70	6.4
		Downstream toe at conduit.					2/	13	21	38	65	77							64												
		Jugged area.					3/	16	24	40	65	74							73												
S-2	2	Okla. Little Wewoka Cr. #17.	E	3.43	7.5YR	5/6	1/	26	31	45	67	78	88	99	100					12	40	17	23	13.8	CL	45.7				2.70	11.4
		Downstream slope. Mid-height.					2/	3	10	28	49	62							32												
		Near conduit. Jugged area.					3/	4	13	28	50	60							42												
S-3		Okla. Little Wewoka Cr. #17.	E	4.20	2.5Y	5/2	1/	14	19	31	52	68	83	97	98	99	100			1.2	24	16	8	14.7	CL	12.3				2.67	14.9
		Upstream slope. Mid-height.					2/	9	16	30	51	66							84												
		Right end of dam near a jug.					3/	11	19	33	52	64							100												
S-4	1	Okla. Caney Coon #2.	N	37	10YR	6/4	1/	30	43	70	89	98								4	40	17	23	10.6	CL	57.7	14	110.5	18.0	2.71	9.5
	*	Stream channel wall, 100 ft.					2/	9	18	45	64	72							42												
		downstream from end of conduit.					3/	10	22	49	64	69							51												
S-5	2	Okla. Caney Coon #2.	E	3.64	10YR	6/3	1/	30	42	68	85	91								1.6	38	17	21	9.7	CL	54.4				2.70	14.0
	*	Downstream toe at conduit.					2/	17	30	53	67	75							71												
							3/	18	30	60	68	72							71												
S-6		Okla. Caney Coon #2.	N	47	10YR	5/4	1/	22	29	43	59	73	83	98	99	100				2	27	14	13	11.5	CL	29.1	8	113.5	14.5	2.70	5.8
	*	Left abutment, 100 ft. from					2/	9	21	36	52	66							72												
		dam.					3/	13	26	45	60	68							90												
S-7.1	4	Okla. Caney Coon #2.	N	45	5YR	4/4	1/	28	30	36	47	53	59	94	99	100				40	31	18	13	13.6	CL	31.4	8	110.0	16.5	2.69	8.0
	*	Left abutment, 100 ft. from					2/		5	8	15	27							17												
		dam.					3/	0	4	7	17	26							13												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ≈8 ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ≈1 ppm).

* Samples selected as best effort to be representative of main borrow pits.

APPENDIX C

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA
Sheet 2 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART		MECHANICAL ANALYSIS – GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT – FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD) MAX. γ_d p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-7.2	4	Okla. Caney Coon #2.	N	55	10YR 6/3	1/	30	39	51	71	81	92	98	100					9	44	21	23	12.9	CL	55.3	12	104.5	20.5	2.71	9.5
	*	Left abutment, 100 ft. from				2/	2	7	17	29	45							18												
		dam.				3/	0	9	19	33	43							23												
S-8	1	Okla. Leader Middle Clear	E	38	2.5Y 6/4	1/	18	23	33	49	59	72	98	99	99	100			1.0	27	15	12	14.4	CL	24.1	7	114.5	15.5	2.68	6.4
		Boggy #15. D.S. slope, near				2/	15	21	30	47	57							91												
		crest, 150'± rt. of conduit.				3/	16	22	36	47	58							96												
		Jugs.																												
S-9	2	Okla. Leader Middle Clear	N	46	2.5Y 7/4	1/	20	25	37	56	67	82	99	100					8	27	15	12	12.8	CL	25.8	8	112.5	15.0	2.69	6.0
	*	Boggy #15. Wall of discharge				2/	3	11	25	43	59							44												
		channel below conduit.				3/	0	10	25	41	49							40												
S-10	3	Okla. Leader Middle Clear	N	5.54	2.5Y 6/6	1/	20	26	34	51	62	75	99	100					9	30	15	15	13.7	CL	28.9				2.70	11.8
	*	Boggy #15. Wall of discharge				2/	2	6	15	21	33							23												
		channel below conduit.				3/	1	9	19	31	41							35												
S-11	1	Okla. Leader Middle Clear	N	44	10YR 6/4	1/	30	40	62	85	93								1.4	37	17	20	11.1	CL	49.6	12	109.5	18.0	2.70	9.3
	*	Boggy #29. Wall of canal				2/	20	31	53	73	86							78												
		between two reservoirs.				3/	24	36	62	76	83							90												
		(Severe erosion gullies)																												
S-12	1	Okla. Upper Clear Boggy #53.	E	5.40	2.5Y 4/2	1/	21	27	38	54	64	77	98	99	100				2.4	29	15	14	12.9	CL	30.3				2.69	9.1
		Mid-height, downstream slope,				2/	8	12	27	43	57							44												
		200 ft. right of conduit.				3/	8	11	33	46	57							70												
		(No erosion on dam)																												
S-13	2	Okla. Upper Clear Boggy #53.	E	32	2.5Y 4/2	1/	16	23	35	54	66	81	99	100					2.5	28	16	12	13.9	CL	24.1	8	111.5	16.0	2.65	9.2
		Same location on upstream slope.				2/	8	15	30	51	66							65												
		(No erosion on dam)				3/	10	19	39	52	60							83												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ______ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ______ ppm).

* See Sheet 1.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA
Sheet 3 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART			MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT: FINES 0.002 mm	FINES 0.005	FINES 0.02	FINES 0.05	#200 0.074	SAND #140 0.105	SAND #60 0.250	SAND #40 0.42	SAND #20 0.84	SAND #10 2.0	SAND #4 4.76	DISPERSION FRACTION < 0.005mm: LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS: LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS: VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD): MAX. γ_d p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-14	1a	Okla. Upper Clear Boggy #50.	E	5.27	2.5Y	5/6	1/	22	29	41	60	74	85	98	99	100				14	30	16	14	12.2	CL	32.6				2.69	7.4
		Undist. samples from bore hole					2/	1	7	16	33	50							24												
		@ lt.end of breach- @ 5' depth					3/	1	7	25	41	50							24												
		(Dam too new for erosion)																													
S-15	1b	" - @ 10' depth	E	5.15	2.5Y	5/4	1/	32	43	55	73	80	88	97	98	99	100			5.5	36	18	18	10.9	CL	48.8				2.71	6.9
							2/	4	13	28	45	60							30												
							3/	6	16	33	46	54							37												
S-16	1c	" - @ 15' depth	E	4.70	2.5Y	6/4	1/	23	28	45	67	80	89	99	100					1.2	31	15	16	13.3	CL	34.0				2.69	5.7
							2/	14	27	41	62	72							76												
							3/	20	26	42	60	71							93												
S-17	1d	" - @ 20' depth	E	5.45	2.5Y	5/4	1/	22	27	41	64	79	89	99	100					1.4	31	15	16	13.5	CL	31.5				2.68	6.3
							2/	12	25	38	58	72							93												
							3/	15	23	40	59	70							85												
S-18	1e	" - @ 25' depth	E	4.20	2.5Y	4/4	1/	30	33	55	74	86								6.5	35	18	17	11.3	CL	45.6				2.69	7.7
							2/	3	11	30	52	66							33												
							3/	3	12	35	54	60							36												
S-19	1f	" - @ 30' depth	N	4.70	2.5Y	6/6	1/	25	29	40	51	60	72	96	98	99	100			9.5	31	14	17	12.0	CL	35.2				2.70	6.2
							2/	3	6	21	30	44							21												
							3/	1	8	25	37	54							28												
S-20	1g	" - @ 35' depth	N	5.19	5Y	6/4	1/	46	57	68	79	86								8.5	51	21	30	8.8	CH	83.4				2.74	8.9
							2/	4	10	25	36	49							18												
							3/	6	8	28	41	50							14												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ________ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ________ ppm).

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA

Laboratory Sample Number	Field Number	Location and Description	Type Deposit E=Emb. N=Nat.	Total Wt. of Sample (lbs.)	Color (Dry) Munsel Chart			Mechanical Analysis: Fines 0.002 mm	0.005	0.02	0.05	#200 0.074	Sand #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	Dispersion Fraction < 0.005mm: Lab Method %	Field Method (Dilution Ratio)	Atterberg Limits LL	PL	PI	SL	Unified Classification	Shrinkage Factors Vol. Shrink %	Lin. Shrink %	Moisture-Density Harvard Min. (Standard) Max. γd p.c.f.	W_o %	G_s	Moisture Content of Fraction < 2mm after Preparation %
S-21	2	Okla. Upper Clear Boggy #50.	E	42	2.5Y	4/4	1/	25	30	49	73	85	93	99	100					1.2	34	16	18	12.5	CL	39.3	10	110.0	17.5	2.69	8.4
		Left wall of breach.					2/	18	28	47	69	82							93												
		Near erosion tunnel.					3/	21	29	48	65	74							97												
S-22	3	Okla. Upper Clear Boggy #50.	E	44	10YR	6/4	1/	24	30	47	71	85	93	99	100					1.2	33	16	17	12.2	CL	39.2	10	111.5	16.5	2.70	6.9
		Left wall of breach. Near ¢.					2/	19	28	44	67	80							93												
							3/	21	28	45	65	73							93												
S-23	1	Okla. Leader Middle Clear	N	45	10YR	6/6	1/	31	38	53	79	93								12.5	40	22	18	15.3	CL	44.1	12	103.0	20.5	2.70	11.1
	*	Boggy #33. Above prin.spwy.					2/	2	9	28	50	63							24												
		level. 200' u.s. of dam.					3/	3	11	29	47	58							29												
		Left valley wall.																													
S-24	2	" "	N	4.20	10YR	5/6	1/	24	27	35	45	55	65	79	83	84	86	91		12.0	35	21	14	15.1	CL	35.1				2.71	18.0
	*						2/	2	3	9	14	17							11												
							3/	0	5	12	13	17							19												
S-25	3	" "	N	3.83	10YR	6/6	1/	28	33	48	68	81	89	98	99	100				2.4	35	21	14	16.1	CL	34.0				2.68	25.2
	*						2/	19	29	42	63	77							88												
							3/	18	28	44	60	71							85												
S-26	1	Okla. Owl Creek #7.	N	47	2.5YR	5/6	1/	43	58	70	83	87								17	49	20	29	6.0	CL	86.1	16.0	105.5	20.0	2.72	14.1
		Just above prin.spwy.level at					2/	1	11	18	21	23							19												
		lake's edge 100' u.s. from dam.					3/	4	10	20	24	25							17												
S-27	2	Okla. Owl Creek #7.	E	46	2.5YR	4/6	1/	26	33	53	67	80	88	99	100					2	32	16	16	12.1	CL	36.6	10.0	112.5	15.5	2.71	8.9
		Downstream toe at conduit.					2/	10	24	40	61	71							73												
		Erosion.					3/	16	25	42	70	73							76												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ______ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ______ ppm).

* See Sheet 1.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA
Sheet 5 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSELL CHART		MECHANICAL ANALYSIS — FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD) MAX. γd p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-28	3	Okla. Owl Creek #7.	N	5.80	2.5YR 4/6	1/	22	32	49	61	74	86	92	92	100				1.6	29	14	15	10.5	CL	36.6				2.76	6.6
	*	Slick spot just below d.s.				2/	15	26	43	56	63							81												
		toe at center of dam.				3/	17	28	44	57	72							88												
S-29	1	Okla. Owl Creek #13.	N	48	10R 4/6	1/	36	69	97	100									1.8	42	22	20	13.2	CL	54.3	12	108.0	19.0	2.77	9.0
	*	Borrow pit upstream on right				2/	21	45	72	77	79							65												
		side.				3/	25	47	75	83	90							68												
S-30	2	Okla. Owl Creek #13.	N	5.65	10R 5/8	1/	42	68	95	100									1.2	41	20	21	13.2	CL	56.1				2.75	18.5
	*	Borrow pit upstream on right				2/	37	61	86	92	93							90												
		side.				3/	39	62	89	96	100							91												
S-31	3	Okla. Owl Creek #13.	N	4.42	5YR 5/6	1/	36	43	56	76	85	91	97	98	99	100			1.6	48	20	28	8.8	CL	76.2				2.69	18.1
	*	Borrow pit upstream on left				2/	13	30	51	70	77							70												
		side.				3/	22	35	55	72	83							81												
S-32	1	Okla. Cherokee Sandy #8a.	N	46	10YR 6/4	1/	37	55	82	90	91								2.3	50	23	27	9.1	CL or CH	76.1	16	100.5	22.5	2.73	13.2
	*	Lake shore just above prin. spwy.				2/	0	8	8	8	8							15												
		level 3[illegible]' u.s. on left side.				3/	0	4	5	6	7							7												
S-33	2	Okla. Cherokee Sandy #8a.	E	51	2.5Y 4/2	1/	33	39	53	73	82	84	87	90	95	100			1.4	53	21	32	10.4	CH	78.5	16	99.5	23.0	2.69	10.1
	*	Near downstream toe. 200 ft.				2/	18	36	56	76	82							92												
		to right of ℄ of breach.				3/	26	38	56	78	89							97												
S-34	3	Okla. Cherokee Sandy #8a.	N	37	5YR 5/3	1/	39	72	90	98	99								1.8	70	32	38	8.7	CH	118.4	20	90.5	30.0	2.75	19.2
	*	Lake shore. Right side.				2/	9	19	26	37	41							26												
		300 ft. upstream from dam.				3/	12	19	29	35	43							26												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ________ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ________ ppm).

* See Sheet 1.

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SOIL MECHANICS LABORATORY DATA
Sheet 6 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART		MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT — FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD) MAX. γ_d p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-35	4	Okla. Cherokee Sandy #8a.	N	3.99	5YR 5/6	1/	47	66	86	91	94								1.2	70	27	43	9.2	CH	135.3				2.71	28.5
	*	Lake shore. Right side.				2/	18	36	54	62	67							55												
		300 ft. upstream from dam.				3/	22	38	60	69	76							58												
S-36	5	Okla. Cherokee Sandy #8a.	N	37	5YR 4/4	1/	55	66	79	89	92								1.4	79	27	72	15	CH	183.8	28	90.0	30.5	2.72	16.6
	*	Discharge channel wall just				2/	46	57	69	74	75							86												
		downstream from conduit.				3/	48	57	70	78	84							86												
S-37	1	Okla. Upper Red Rock #42.	N	39	10R 4/6	1/	25	28	42	71	86								25	32	19	13	16.6	CL	28.1	9	108.0	17.5	2.67	8.7
	*	Lake shore. Left side.				1/		5	14	37	49							18												
		100 ft. upstream from dam.				3/	1	2	11	26	43							7												
S-38		Okla. Upper Red Rock #42.	E	5.30	10R 4/6	1/	22	26	45	77	91								5.5	29	17	12	15.4	CL	26.4				2.69	17.5
		Upstream slope @ water level -				2/	1	13	23	39	46							50												
		right end. (Near jugs)				3/	8	13	30	51	65							50												
S-39	3	Okla. Upper Red Rock #42.	E	5.43	5YR 3/8	1/	25	30	49	78	92								2.0	32	17	15	15.7	CL	31.5				2.69	19.8
		Upstream slope @ water level -				2/	7	19	40	60	70							63												
		center. (Near jugs)				3/	13	22	42	68	82							73												
S-40	1	Okla. Upper Red Rock #48.	N	31	10R 3/6	1/	40	49	62	78	84	86	87	88	91	94	96		1.6	52	22	30	10.7	CH	86	17	100.0	22.5	2.73	10.7
	*	Lake shore. Left side.				2/	19	32	45	60	66							65												
		100 ft. upstream from dam.				3/	26	34	47	62	71							69												
S-41	2	Okla. Upper Red Rock #48.	N	4.45	2.5YR 4/8	1/	28	32	47	75	90								1.6	39	16	23	12.7	CL	51				2.68	15.3
	*	Emergency spillway channel.				2/	14	26	42	62	72							81												
		Erosion gullies.				3/	21	28	44	66	82							88												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = __________ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = __________ ppm).

* See Sheet 1.

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SOIL MECHANICS LABORATORY DATA
Sheet 7 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART		MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT: FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD) MAX. γd p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-42	1	Okla. Wister Dam.	E	49	10YR 5/4	1/	22	39	71	91	98								2.4	32	22	10	19.2	CL	24.6	7	102.0	20.0	2.69	9.3
		D.S. slope. Right half. Above				2/	7	23	51	65	69							59												
		berm. Samples near larger jugs.				3/	9	27	51	67	82							69												
		Over 1000 ft. dam length.																												
S-43	2	" "	E	37	10YR 5/6	1/	18	32	64	87	94								4	32	21	11	18.6	CL	24.6		102.0	20.0	2.69	8.2
						2/	1	14	43	60	68							44												
						3/	3	15	45	65	78							47												
S-44	3	" "	E	48	10YR 6/4	1/	25	40	68	84	93								1.8	32	20	12	14.6	CL	35.3	9	100.5	21.5	2.70	8.4
						2/	8	25	50	63	68							63												
						3/	12	25	50	66	77							63												
S-45	4	" "	E	38	10YR 6/4	1/	22	39	71	87	95								2.0	33	21	12	19.2	CL	23.8	7	101.5	21.0	2.71	14.0
						2/	8	26	52	66	70							67												
						3/	10	26	53	68	79							67												
S-46	1E	Venezuelan Dikes.	E	11.92	5YR 5/6	1/	35	53	77	93	95								1.6	42	23	19	18.7	CL	42.5	11	98.5	24.0	2.80	12.9
		Samples from crest of dike in				2/	21	41	63	74	78							77												
		zone of worst erosion & jugs.				3/	24	42	69	81	91							79												
		Near kilometer 63.																												
S-47	2E	" "	E	12.10	5YR 5/6	1/	31	47	75	91	94								1.2	38	23	15	18	CL	37.8	10	101.0	22.5	2.76	13.2
						2/	21	43	64	76	81							91												
						3/	26	43	67	81	90							91												
S-48	3E	" "	E	11.97	5YR 5/6	1/	36	54	77	92	99								1.2	42	23	19	17.4	CL	45.8	11	99.5	23.5	2.75	10.5
						2/	26	47	62	77	82							87												
						3/	33	52	72	83	91							96												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ________ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ________ ppm).

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SOIL MECHANICS LABORATORY DATA
Sheet 8 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART		MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT — FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD) MAX. γd p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-49	4E	Venezuelan Dikes.	E	11.60	5YR 5/6	1/	31	43	64	81	93								2.6	36	22	14	16.6	CL	36.6	10	101.5	21.5	2.73	12.3
		Samples from crest of dike in				2/	13	26	44	64	72							60												
		zone of worst erosion & jugs.				3/	15	28	45	64	74							65												
		Near kilometer 63.																												
S-50	5E	" "	E	12.15	5YR 5/6	1/	23	33	51	65	77	90	99	100					2.8	32	19	13	16.4	CL	30.2	8	106.0	19.0	2.73	13.1
						2/	9	23	40	59	65							70												
						3/	14	23	41	58	73							70												
S-51	6E	" "	E	11.88	10YR 5/6	1/	56	80	95	100									4	63	29	34	16.9	CH	81.1	16	93.5	28.0	2.76	12.1
						2/	7	18	38	49	57							23												
						3/	14	31	50	62	71							39												
S-52	1N	Venezuelan Dikes.	E	11.70	5YR 5/6	1/	35	48	70	89	96								2.2	42	21	21	14.8	CL	52.1	12	103.5	21.0	2.74	12.5
		Dike crest in zone of little				2/	13	28	52	72	79							58												
		or no erosion. Near kilometer				3/	21	35	58	76	85							73												
		58.5																												
S-53	2N	" "	E	11.95	7.5YR 5/4	1/	45	67	88	95	100								4	53	26	27	16.6	CH	67.1	12	97.0	25.5	2.74	12.7
						2/	10	22	45	59	65							33												
						3/	11	27	49	62	70							40												
S-54	3N	" "	E	11.31	7.5YR 5/6	1/	44	73	98	100									3	51	27	24	17.6	CH	59.4	11	94.5	26.5	2.78	12.7
						2/	5	11	51	62	68							15												
						3/	7	21	52	63	71							29												
S-55	4N	" "	E	11.61	7.5YR 5/6	1/	55	84	100										1.4	60	28	32	17.2	CH	78.7	14	93.5	27.0	2.76	14.0
						2/	41	66	83	89	93							79												
						3/	47	72	86	91	96							86												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ______ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ______ ppm).

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA
Sheet 9 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART		MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT — FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD) MAX. γ d p.c.f.	w_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-56	5N	Venezuelan Dikes.	E	11.92	7.5YR 5/8	1/	54	79	96	100									6.5	57	26	31	16.6	CH	74	15	93.5	27.5	2.76	13.7
		Dike crest in zone of little				2/	7	15	47	59	69							19												
		or no erosion. Near kilometer				3/	5	19	48	56	62							24												
		58.5																												
S-57	6N	" "	E	11.68	5YR 5/6	1/	26	37	57	77	84	92	98	99	100				3	33	20	13	17.6	CL	28.8	8	105.0	19.5	2.74	13.1
						2/	8	16	42	58	71							43												
						3/	8	21	45	61	74							57												
S-58	1	Calif. Stockton Creek Dam.	E	46	7.5YR 5/6	1/	9	17	30	41	45	48	55	57	62	69	76		5.5	31	23	8	20.2	SC or SM	18.3	6	108.5	19.0	2.89	3.6
	*	Downstream slope. Center of				2/	1	3	13	19	24							18												
		dam. Mid-height.				3/	0	4	11	20	28							24												
		(Dam failed by piping)																												
S-59	2	Calif. Stockton Creek Dam.	N	29	5YR 5/6	1/	23	34	61	82	92								7	32	21	11	17.8	CL	28	8	105.5	20.5	2.90	4.7
	*	Borrow area. Upstream.				2/	3	5	21	39	51							15												
		Left valley wall.				3/	1	8	20	39	50							24												
S-60	3	Calif. Stockton Creek Dam.	N	32	5YR 5/8	1/	13	23	43	58	62	67	74	77	82	87	92		5	29	22	7	20.8	CL-ML	16.6	6	107.0	19.5	2.85	3.7
	*	Borrow area. Near spillway.				2/	3	5	20	30	39							22												
						3/	0	6	18	29	38							26												
S-61	1	Hills Creek Dam. Samples from	E	4.12	10YR 4/3	1/	27	36	48	58	60	62	67	71	77	82	87		1.6	45	26	19	16.3	CL	53.6				2.71	27.6
	*	large dia. auger boring in				2/	10	13	26	32	33							36												
		crest - @ 15'-21' depth				3/	7	15	23	28	32							42												
		(Piping through central core)																												
S-62	2	" - @ 25'-31' depth	E	4.73	10YR 5/4	1/	20	25	37	46	49	51	57	62	69	75	83		1.2	40	24	16	16.4	SC	45.8				2.74	25.0
	*					2/	11	20	30	38	43							80												
						3/	9	14	21	26	32							56												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ___ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ___ ppm).

* See Sheet 1.

U. S. DEPARTMENT OF AGRICULTURE
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SOIL MECHANICS LABORATORY DATA
Sheet 10 of ___

Laboratory Sample Number	Field Number	Location and Description	Type Deposit E=Emb. N=Nat.	Total Wt. of Sample (lbs.)	Color (Dry) Munsel Chart			Mechanical Analysis – Grain size distribution expressed as percent finer by dry weight – Fines 0.002 mm	0.005	0.02	0.05	#200 0.074	Sand #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	Dispersion Fraction < 0.005mm – Lab Method %	Field Method (Dilution Ratio)	Atterberg Limits LL	PL	PI	SL	Unified Classification	Shrinkage Factors Vol. Shrink %	Lin. Shrink %	Moisture-Density Harvard Min. (Standard) Max. γd p.c.f.	W_o %	G_s	Moisture Content of Fraction < 2mm after Preparation %
S-63	3	Hills Creek Dam. Samples from	E	4.13	10YR	5/4	1/	25	34	49	57	61	64	70	73	79	84	89		1.6	42	23	19	16.2	CL	46.3				2.72	25.3
	*	large dia. auger boring in					2/	13	19	34	39	43							56												
		crest - @ 37'-46' depth					3/	13	18	30	36	42							53												
S-64	4	" - @ 50'-60' depth	E	11.40	10YR	5/4	1/	26	36	50	59	61	64	69	73	79	83	89		1.4	42	23	19	15.2	CL	51	11	97.0	25.0	2.72	26.7
	*						2/	18	27	39	44	48							75												
							3/	15	25	36	42	47							69												
S-65	1 & 2	Lost Creek Dam.	N	2.78	10YR	4/3	1/	29	41	60	71	72	74	81	85	91				2.4	41	27	14	14.1	ML	52.4				2.72	
	*	Borrow pit samples from same					2/	8	22	35	47	56							54												
		hole @ depths 10 & 24 ft.					3/	12	25	40	50	61							61												
		(Dam in design stage)																													
S-66		(Not used)					1/																								
							2/																								
							3/																								
S-67	1	Norway, Hyttejuvet Dam.	N	1.85	2.5Y	7/0	1/	8	14	29	37	44	47	55	59	65	72	80		1.8	19	15	4	13.7	SC-SM	12.3				2.83	
	*	Borrow pit sample held in					2/	2	7	22	31	40							50												
		Oslo Lab. (Piping thru					3/	1	10	20	32	42							71												
		central core)																													
S-68.1	1	England, Balderhead Dam.	N	5.93	2.5Y	4/2	1/	24	34	47	58	64	69	77	80	83	87	90		2.4	35	19	16	13.3	CL	42.5	10	109.0	17.0	2.69	10.8
	*	Borrow pit sample. CH 1385,					2/	8	17	31	42	49							50												
		20' below crest. (Piping					3/	8	18	35	43	54							53												
		through central core)																													
S-68.2	2	England, Balderhead Dam.	N	3.82	10YR	4/2	1/	18	24	33	42	47	52	66	72	78	81	86		50	27	14	13	13.3	SC	30.4				2.69	10.4
	*	Borrow pit sample. CH 1375,					2/	2	3	23	35	38							13												
		90'-100' below crest.					3/	6	15	27	36	44							62												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ______ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ______ ppm).

* See Sheet 1.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA
Sheet 11 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART			MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT: FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm: LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS: LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS: VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD): MAX. γd p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-69	1	Utah, Mill Site Dam.	N	22.4	2.5Y	6/2	1/	10	14	22	39	62	81	85	86	88	92	95		13	19	0	0	18.4	ML	6.0	1	108.0	14.5	2.72	1.2
	*	Typical core material.					2/	1	3	16	25	47							21												
		(Dam under construction)					3/	1	3	15	26	50							21												
S-70	2	Utah, Mill Site Dam.	N	19.5	5Y	6/2	1/	12	17	27	45	63	77	81	83	84	87	91		14	19	17	2	12.7	ML	13.6	6	114.5	13.5	2.71	1.2
	*	Typical core material.					2/	2	4	14	24	49							23												
							3/	0	5	17	28	52							29												
S-71	3	Utah, Mill Site Dam.	N	22	5Y	6/2	1/	13	19	29	48	70	84	89	91	92	93	94		10	19	17	2	15.6	ML	8.9	2	113.0	14.0	2.69	1.1
	*	Typical core material.					2/	1	3	14	28	47							16												
							3/	0	6	14	34	59							32												
S-72	1	Lake Yosemite Dam.	E	34	10YR	5/6	1/	18	29	43	59	65	72	83	86	90	92	93		6.5	38	23	15	12.6	CL	45.5	10	100.0	22.0	2.69	7.6
	*	Downstream slope. Mid-height.					2/		3	9	19	27							10												
		Near brick conduit. 200' apart.					3/	0	6	11	19	30							21												
S-73	2	" "	E	32	10YR	5/6	1/	21	31	47	60	68	73	83	87	92	94	96		7	38	21	17	12.5	CL	47.9	10	99.5	22.0	2.67	7.9
	*	(Dam withstood leakage many					2/	1	3	10	22	31							10												
		years with no piping)					3/	0	5	12	24	33							16												
S-74	1	Okla. Frogville #2.	N	42	7.5YR	6/6	1/	23	28	40	64	76	84	97	98	99	100			13	37	18	19	10.7	CL	52.7	12	104.0	19.5	2.66	9.8
		(From crest and slopes.					2/	0	3	9	21	30							11												
		Composite of typical lime					3/	0	3	9	24	38							11												
		treated soil)																													
S-75	2	Okla. Frogville #2	N	35	10YR	7/4	1/	20	25	33	50	61	73	99	100					6	31	15	16	10.5	CL	41.0	9	110.0	17.0	2.66	6.6
	*	(From soil outcrop. 200 feet					2/	2	9	17	36	50							36												
		downstream in vicinity of					3/	2	9	21	39	56							36												
		erosion gully)																													

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ___ ppm).
3/ Dispersion test hydrometer using SSL distilled water (total dissolved solids = ___ ppm).

* See Sheet 1.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA
Sheet 12 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART		MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT — FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD) MAX. γd p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-76.1	1	Ark. Caney Creek #7.	E	9.95	10YR 5/4	1/	10	13	48	92	99	100							1.2	28	23	5	20.7	ML	12.3	2	99.5	19.5	2.67	18.9
		Shallow pits in u.s. slope just				2/	3	10	49	85	88							77												
		above perm. pool near prominent				3/	7	12	46	84	90							92												
		jugs.																												
S-76.2	2	" "	E	9.75	10YR 5/4	1/	10	13	45	93	98								1.2	28	22	6	20.7	CL-ML	11.4	2	100.5	19.0	2.67	19.6
						2/	3	8	45	80	84							62												
						3/	4	9	42	78	87							69												
S-77		Ark. Caney Creek #7.	E	11.70	10YR 5/3	1/	16	21	59	93	100								1.0	32	22	10	19.7	CL	20.8	5	100.0	20.5	2.67	20.6
		Blanket on left abutment				2/	14	21	65	91	98							100												
		where had severe jugging.				3/	12	21	58	90	94							100												
S-78.1	1	Wynne, Ark.--Hall Reservoir.	E	11.50	10YR 5/6	1/	15	21	58	94	100								2.6	33	23	10	18.5	CL or ML	24.6	4	98.5	20.5	2.67	15.9
		(Samples from downstream				2/	3	11	51	78	82							52												
		slope near jugs)				3/	7	13	50	74	79							62												
S-78.2	2	" "	E	10.75	10YR 6/4	1/	15	20	60	94	100								1.4	34	22	12	19.5	CL	25.7	5	98.5	22.0	2.68	16.4
						2/	7	14	56	84	86							70												
						3/	7	16	53	85	92							80												
S-79	1	Ark. Caney Creek #6.		13.75	10YR 5/4	1/	13	18	54	89	91	91	92	93	94	95	96		1.0	32	22	10	19.8	CL	20.8	5	100.5	20.0	2.67	19.5
		From wall of large jug on				2/	10	14	55	84	86							78												
		d.s. slope. Mid-length.				3/	10	16	53	82	87							89												
S-80	1	Ill. Washington County Dam.	E	10.40	2.5Y 6/6	1/	24	31	50	69	75	79	88	94	98	100			4.5	35	16	19	9.2	CL	52.2	10	109.0	18.0	2.69	17.6
	*	3 locations from shallow pits				2/	3	5	17	17	24							16												
		on d.s. slope. 3 different				3/	0	5	14	17	23							16												
		elev. & sta. All similar in appearance.																												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ________ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ________ ppm).

* See Sheet 1.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA
Sheet 13 of ___

LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART		MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT: FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm: LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS: LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS: VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD): MAX. γ_d p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-81	2	Ill. Washington County Dam.	E	10.90	2.5Y 5/6	1/	26	32	53	72	78	82	91	96	99	100			12	37	17	20	11.1	CL	48.8	12	107.0	19.0	2.69	15.5
	*	3 locations from shallow pits				2/	1	3	14	14	15							9												
		on d.s. slope. 3 different				3/	0	4	11	15	21							13												
		elev. & sta. All very similar																												
		in appearance.																												
S-82		" "	E	9.30	2.5Y 5/6	1/	26	32	52	68	73	78	90	96	99	100			9	37	15	22	9.0	CL	54.3	12	107.0	17.5	2.73	15.1
	*	(No jugging. Dam failed by				2/		3	13	13	17							9												
		piping similar to Okla.				3/	0	5	14	17	26							16												
		SCS failures)																												
S-83	4	Ill. Washington County Dam.	N	6.43	2.5Y 5/6	1/	20	26	40	54	58	65	86	95	99	100			3	29	14	15	12.5	CL	32.7				2.69	17.6
	*	Left valley wall, just at u.s.				2/	3	5	17	21	23							19												
		end of em.spwy. Probably				3/	1	8	17	25	30							31												
		glacial drift formation.																												
S-84	5	" "	N	6.34	2.5Y 4/6	1/	36	43	59	71	75	79	89	95	99	100			12	46	19	27	10.1	CL	69.6				2.71	19.0
	*					2/			6	6	6							2												
						3/	0	4	6	6	7							9												
S-85	1	Miss. Perry Creek #3.	E	56	7.5YR 7/8	1/	23	27	61	94	96								4.5	38	21	17	12.9	CL	47.9	10	99.5	21.5	2.69	18.9
		From wall of large jug on				2/	3	6	14	69	75							22												
		crest.				3/	11	21	52	82	86							78												
S-86	1	Miss. Big Sand #10.	1	52	7.5YR 7/8	1/	22	27	54	78	83	85	95	99	100				7	34	17	17	12.5	CL	42.6	11	105.5	18.5	2.68	16.8
		Adjacent large jug on down-				2/	3	9	25	43	49							33												
		stream slope near top.				3/	6	14	40	54	62							52												
S-87	1	Miss. Big Sand #8.	E	63	7.5YR 7/6	1/	17	21	47	71	77	80	95	99	100				1.0	28	17	11	15.9	CL	20.3	6	109.0	17.0	2.67	9.9
		Upstream slope near crest.				2/	12	20	44	67	75							95												
		Sample from immediate vicinity				3/	11	21	48	65	74							100												
		of jugs.																												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ______ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ______ ppm).

* See Sheet 1.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL MECHANICS LABORATORY DATA
Sheet 14 of ___

Laboratory Sample Number	Field Number	Location and Description	Type Deposit E=Emb. N=Nat.	Total Wt. of Sample (lbs.)	Color (Dry) Munsel Chart		Mechanical Analysis, Fines: 0.002 mm	0.005	0.02	0.05	#200 0.074	Sand: #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	Dispersion Fraction < 0.005mm: Lab Method %	Field Method (Dilution Ratio)	Atterberg Limits: LL	PL	PI	SL	Unified Classification	Shrinkage Factors: Vol. Shrink %	Lin. Shrink %	Moisture-Density Harvard Min. (Standard): Max. γd p.c.f.	W_o %	G_s	Moisture Content of Fraction < 2mm After Preparation %
S-88	1	Miss. Big Sand #8.	E	77	10YR 6/6	1/	16	21	47	71	77	80	95	99	100				1.2	30	17	13	13.9	CL	29.3	7	107.0	17.5	2.67	14.1
		Downstream berm near conduit.				2/	12	18	52	66	75							86												
		Sample from immediate vicinity				3/	12	20	43	68	76							95												
		of jugs.																												
S-89	1	Miss. Big Sand #8.	E	72	10YR 6/8	1/	20	24	52	74	82	84	93	98	100				1.4	34	17	17	14.2	CL	35.4		107.0	18.0	2.67	17.8
		Downstream slope at right end.				2/	14	21	36	70	77							88												
		Sample from immediate vicinity				3/	14	21	48	73	76							88												
		of jugs.																												
S-90	1	Miss. Potacocawa #3.	E	59	10YR 5/6	1/	16	17	50	83	93								2.5	29	20	9	18.5	CL	19.4	4	105.5	17.5	2.68	19.0
		Downstream slope in bad area				2/	7	8	33	45	62							47												
		of jugging near conduit.				3/	3	13	33	63	69							77												
S-91	2	Miss. Potacocawa #3.	E	55	10YR 6/6	1/	19	23	51	85	90								2.4	32	20	12	17.3	CL	27.9	7	104.0	19.0	2.65	19.8
		Upstream slope near jugs				2/	6	7	23	43	52							30												
		100 ft. to right of conduit.				3/	7	15	43	69	76							65												
S-92	1	L.A.,Calif.-Baldwin Hills Res.	N	36	2.5Y 6/2	1/	10	13	27	62	79	94	100						3.0	32	27	5	24.4	ML	12.6	3	93.5	24.5	2.72	18.2
		Fdn mat'l fm rt.wall of breach				2/	3	8	34	48	69							62												
		about 6' abv level of res.bottom.				3/	0	8	27	50	71							62												
		Mixture of thinly-bedded cohesive																												
		soil & fine, silty sand.																												
S-93	2	L.A.Calif.-Baldwin Hills Res.	N	28	2.5Y 7/4	1/	17	25	48	76	87								2.8	44	25	19	23.6	CL	32.6	8	90.0	28.5	2.73	19.1
		Same as above except taken fm				2/	0	8	16	54	65							32												
		single 12" thick layer of typical				3/	1	13	33	55	69							52												
		silty clay (or clayey silt) of																												
		the formation.																												
S-94	3	L.A.Calif.-Baldwin Hills Res.	E	32	2.5Y 4/2	2/	23	27	43	56	66	73	85	91	97	100			13	37	18	19	12.7	CL	45.7	5	106.0	19.0	2.71	6.9
		Typical compacted clay reservoir				2/	0	3	20	29	41							11												
		lining taken from rt.wall of				3/	0	6	22	35	49							22												
		breach about 20' above reservoir																												
		bottom.																												

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ________ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ________ ppm).

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LABORATORY SAMPLE NUMBER	FIELD NUMBER	LOCATION AND DESCRIPTION	TYPE DEPOSIT E=EMB. N=NAT.	TOTAL WT. OF SAMPLE (lbs.)	COLOR (DRY) MUNSEL CHART		MECHANICAL ANALYSIS GRAIN SIZE DISTRIBUTION EXPRESSED AS PERCENT FINER BY DRY WEIGHT — FINES 0.002 mm	0.005	0.02	0.05	#200 0.074	SAND #140 0.105	#60 0.250	#40 0.42	#20 0.84	#10 2.0	#4 4.76	DISPERSION FRACTION < 0.005mm LAB METHOD %	FIELD METHOD (DILUTION RATIO)	ATTERBERG LIMITS LL	PL	PI	SL	UNIFIED CLASSIFICATION	SHRINKAGE FACTORS VOL. SHRINK %	LIN. SHRINK %	MOISTURE-DENSITY HARVARD MIN. (STANDARD) MAX. γd p.c.f.	W_o %	G_s	MOISTURE CONTENT OF FRACTION < 2mm AFTER PREPARATION %
S-95	1	Calif.Dept.Water Res.--Cedar	E	19	7.5YR 5/2	1/	47	62	82	91	95								12	44	20	24	11.2	CL	65.0	13	99.0	23.5	2.77	3.0
		Springs Dam. Typical clay				2/	0	4	21	33	44							6												
		core material-under construction.				3/	0	8	21	38	51							13												
S-96	2	Calif.Dept.Water Res.--Cedar	E	19.	7.5YR 5/4	1/	43	57	75	81	86								11	42	22	20	11.2	CL	61.2	12	98.5	23.5	2.82	1.7
		Springs Dam. Hauled in for				2/	0	5	25	36	52							9												
		20 miles from the site.				3/	0	8	25	43	58							14												
S-97	1	New Zealand, Matahina Dam.		4.18	2.5Y 8/8	1/	21	30	47	64	69	75	86	89	94	100			2.4	27	21	6	18.2	CL ML	21.8				2.67	3.7
	*	Core material. (Piping				2/	3	11	23	36	45							37												
		damage to central core dam)				3/	2	13	26	38	47							43												
S-98		San Antonio, Tex., Braunig Lake		0.15	10YR 6/4	1/	2	5	7	8	10	17	97	100																
		Dam (Failed 1/70). 30.5-31.				2/																								
		"A" sand 3" above failure. CB-1.				3/																								
S-99		San Antonio, Tex., Braunig Lake		1.52	10YR 4/1	1/	54	76	98	100																				20.0
		Dam (Failed 1/70). 30.5-31.				2/	3	13	26	35	41							17												
		Clay in failure plane. CB-1.				3/																								
		(Dam failed by shear slide																												
		in suspected dispersive				1/																								
		clay layer in foundation)				2/																								
						3/																								
						1/																								
						2/																								
						3/																								

1/ Gradation of fine fraction by SML hydrometer method.
2/ Dispersion test (hydrometer) using SML demineralized water (total dissolved solids = ______ ppm).
3/ Dispersion test (hydrometer) using SSL distilled water (total dissolved solids = ______ ppm).

* See Sheet 1.

APPENDIX D

SOIL CHEMICAL AND MINERALOGICAL TEST RESULTS AND RESERVOIR WATER ANALYSES

SOIL CHEMICAL AND MINERALOGICAL TEST RESULTS AND RESERVOIR WATER ANALYSES

(See Appendix B for descriptions of dams from which samples were taken.)

The tests were performed following procedures as described in the manual, "Soil Survey Laboratory Methods and Procedures for Collecting Soil Samples", Soil Survey Investigations Report No. 1, Soil Conservation Service, Washington, D. C., 1967. In each column of tabulated test results the test procedure employed is indicated by the code number in the column heading; e.g., the symbol 6N2e, in the heading of the second column of tabulated results, refers to the test procedure described in Paragraph 6N2e of the Manual.

UNITED STATES DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE - Soil Survey Laboratory

1325 N Street, 4th Floor, Lincoln, Nebraska 68508

JECT: Testing program on special samples for piping and jugging study

DATE: April 22, 1971

TO: Rey S. Decker, Head,
Soil Mechanics Unit, SCS
800 J Street
Lincoln, Nebraska 68508

We have completed the chemical and mineralogical analyses for the samples of soil (102) and water (16) outlined under Item III of the testing program referred to in your letter to Mr. James L. Sherard of February 17, 1971.

Attached are two copies of the laboratory work sheets. We are short of clerical help at the moment and did not type the data. This can be done next week if there is a need--give us a call. Most of these data were transmitted to you informally on April 7.

Dr. Lynn's discussion of the mineralogical analyses for the nine samples referred to in your memorandum of March 4, 1971, is given at the end of this memorandum. Mr. Juve's comments on the chemical data follow.

* * *

Of the soil samples checked for effervescence, some 38 indicated the presence of calcium carbonate. In most cases this fact would probably account for the sum of bases exceeding the cation exchange capacity, expecially if this difference exceeds about ten percent. A case in point is that of sample S-67 (71L173) which appears to have an exceedingly low CEC value. All bases but calcium are very low. Effervescence with acid is readily observable with a binocular microscope indicating the presence of calcium carbonate. Solubility of this carbonate in ammonium acetate could easily account for the discrepancy in sum of bases and CEC values.

In addition, a few samples contain soluble sulfates, ranging from a trace in some to appreciable amounts in about three of those samples checked. The latter, S-69 (71L176), S-70 (71L177), and S-71 (71L178), obviously contain gypsum since the calcium values exceed the other bases by nearly a factor of ten.

The water samples are all very low in salts with the highest conductivity being only 0.41 mmhos/cm. The anions are mostly bicarbonates with a few samples containing measurable amounts of chloride. The difference

between the measured anions and total soluble salts based upon the electrical conductivity can probably be attributed to sulfates. A qualitative check of two samples which had the largest difference between measured anions and conductivity, S-201 (71L204) and S-211 (71L214), indicated a faint test for sulfates.

* * *

There is no clear-cut evidence from the samples analyzed that clay mineralogy has an effect on piping or jugging. The two from Venezuela have very similar clay mineralogy. Information on the other samples does not indicate samples of stable and piped or jugged material were collected for comparison. Comments on individual samples follow.

S-1 (71L094) The clay contains small to moderate amounts of kaolinite and mica. A moderate amount of a 2:1 layer silicate complex is present. The complex is mostly mica-vermiculite. Some smectite[1/] components may be included.

S-11 (71L105) The clay contains moderate amounts of mica and kaolinite, small vermiculite component, and traces of quartz and feldspar.

S-74 (71L108) The clay is dominated by a 2:1 layer silicate that is a low charge vermiculite or a high charge smectite. Small amounts of kaolinite and mica are present.

S-42 (71L138) The clay contains a moderate amount of kaolinite, a small to moderate amount of mica plus a moderate amount of a 2:1 layer silicate complex. The complex contains components of mica, vermiculite, probably smectite (high charge) and a chlorite-interlayer material.

S-48 (71L144) The clay contains moderate to abundant kaolinite and moderate amounts of mica and vermiculite. A trace of chlorite-interlayer material is present.

S-54 (71L150) The clay contains moderate to abundant kaolinite, a moderate amount of mica and a small amount of vermilulite.

S-79 (71L184) The clay contains small amounts of mica and smectite plus traces of kaolinite and quartz. The smectite is somewhat poorly ordered; the other minerals are well ordered.

1/ Smectites are 2:1 layer silicates that expand upon solvation with glycerol. Montmorillonite is one of several members of the smectite group.

S-86 (71L191) The clay contains a moderate amount of kaolinite, a trace of mica, and a small amount of a mica-vermiculite-chlorite intergrade complex. The kaolinite is well ordered; the other minerals are somewhat poorly ordered.

S-90 (71L195) The clay contains moderate amounts of mica, kaolinite, and smectite. A vermiculite component is likely included with the smectite. Minerals are well ordered.

Robert B. Grossman, Head
Soil Survey Laboratory

Attachments

cc:
G. D. Smith
E. J. Pedersen

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10-64

APR 22 1971

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Soil Mechanics Laboratory

SOIL Soil Piping Study SOIL Nos. LOCATION

SOIL SURVEY LABORATORY Lincoln, Nebraska LAB. Nos. 71L094-71L105

GENERAL METHODS: 1B1b, 2A1, 2B

	SML No.	Extractable bases 5B4a					6H1a	Cat. Exch. Cap.		Water extract from saturated paste 8A1									
		6N2e Ca	6O2d Mg	6P2a Na	6Q2a K	Sum	Ext. Acidity	5A3a Sum Cations	5A6a NH_4OAc	6N1b Ca	6O1b Mg	6P1b Na	6Q1b K		6I1a CO_3	6J1a HCO_3	6K1a Cl	6L1a SO_4	8A1a Electrical conductivity
		meq/100 g								meq/liter									mmho/cm
294	S-1	6.5	5.7	1.3	0.4	13.9			13.3	1.2	0.9	3.4	0.2						0.73
295	S-2	5.4	5.7	0.9	0.4	12.4			13.8	0.2	0.2	2.4	tr						0.37
296	S-3	5.4	2.7	0.3	0.2	8.6			9.9	0.6	0.4	1.3	0.1						0.31
297	S-4	9.0	8.0	1.6	0.4	19.0			18.3	3.0	3.4	8.4	0.1						1.58
298	S-5	8.1	7.2	1.0	0.7	17.0			16.9	1.1	1.2	3.6	0.2						0.70
299	S-6	6.4	4.7	0.8	0.3	12.2			11.4	0.8	0.7	3.6	tr						0.58
100	S-7.1	0.3	0.6	0.1	0.2	1.2			11.0	0.2	0.1	0.2	0.1						0.10
101	S-7.2	0.4	4.8	0.4	0.3	5.9			20.5	tr	tr	0.5	tr						0.07
102	S-8	7.9	4.9	4.4	0.3	17.5			12.2	0.4	0.3	14.9	tr						1.54
103	S-9	7.7	3.3	0.9	0.2	12.1			11.4	6.0	2.7	9.5	0.1						2.01
104	S-10	10.7	3.3	0.4	0.3	14.7			13.6	0.6	0.2	1.3	tr						0.25
105	S-11	12.7	6.1	2.7	0.5	22.0			17.6	0.9	0.5	8.8	tr						0.96

SML No.	8D1 Resistivity ohms-cm	5D2 ESP Pct.	5E SAR	8D5 Total Sol. Salts me/1	8C1b pH Sat. Paste	8A Water at Sat. Pct.	
S-1	2400	8	3	5.7	6.6	44.5	
S-2	4000	6	5	2.8	5.2	54.8	
S-3	6800	3	2	2.4	5.4	28.3	
S-4	1200	6	5	14.9	5.6	51.8	
S-5	1800	5	3	6.1	5.4	43.4	
S-6	2900	6	4	5.1	6.4	33.9	
S-7.1	42000	1	1	0.6	4.5	35.0	
S-7.2	11000	2		0.5	4.2	51.8	
S-8	1500	31	25	15.6	7.4	40.1	
S-9	1400	5	5	18.3	4.8	34.5	
S-10	4600	3	2	2.1	5.3	36.4	
S-11	1500	13	11	10.2	7.6	45.2	

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U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL Soil Mechanics Laboratory
Soil Piping Study
SOIL Nos. ____ LOCATION ____

SOIL SURVEY LABORATORY Lincoln, Nebraska LAB. Nos. 71L106-71L117

GENERAL METHODS: 1B1b, 2A1, 2B

	SML No.	Extractable bases 5B4a: 6N2e Ca	6O2d Mg	6P2a Na	6Q2a K	Sum	6H1a Ext. Acidity	Cat. Exch. Cap.: 5A3a Sum Cations	5A6a NH_4OAc	Water extract from saturated paste 8A1: 6N1b Ca	6O1b Mg	6P1a Na	6Q1b K		6I1a CO_3	6J1a HCO_3	6K1a Cl	6L1a SO_4	8A1a Electrical conductivity
		meq/100 g								meq/liter									mmho/cm
106	S-12	14.2	4.7	4.3	0.3	23.5			13.9	30.0	17.5	56.5	0.1						6.99
107	S-13	15.2	3.0	1.6	0.3	20.1			13.2	2.5	0.9	11.7	0.1						1.25
108	S-74	47.8	2.0	0.3	0.4	50.5			17.7	3.9	0.3	1.0	0.1						0.51
109	S-75	3.9	4.5	0.6	0.2	9.2			14.3	0.2	0.1	1.5	tr						0.23
110	S-14	12.2	2.7	0.6	0.4	15.9			14.1	3.2	0.9	3.7	0.1						0.78
111	S-15	24.9	5.1	1.9	0.4	32.3			17.5	2.0	0.9	9.9	0.1						1.14
112	S-16	8.8	4.0	3.6	0.4	16.4			15.0	1.6	0.8	17.1	tr						2.33
113	S-17	8.3	3.9	1.8	0.3	14.3			14.6	1.7	1.0	11.7	0.1						1.55
114	S-18	18.8	4.4	1.3	0.4	24.9			18.1	6.8	2.7	9.3	0.1						1.80
115	S-19	13.2	2.4	0.7	0.4	16.7			14.8	4.3	1.0	4.6	0.1						1.04
116	S-20	28.1	4.5	1.6	0.6	34.8			24.0	3.6	0.9	6.0	0.1						1.12
117	S-21	13.2	4.9	4.8	0.4	23.3			17.4	3.1	1.8	27.5	0.1						3.34

SML No.	8E1 Resistivity ohms-cm	5D2 ESP Pct.	5E SAR	8D5 Total Sol. Salts me/1	8C1b pH Sat. Paste	8A Water at Sat. Pct.	
S-12	530	15	12	104.1	7.4	38.3	
S-13	1700	9	9	15.2	7.5	37.9	
S-74	2000	2	1	5.3	8.0	43.3	
S-75	4300	4	4	1.8	4.0	36.6	
S-14	1400	3	3	7.9	6.5	36.7	
S-15	1500	9	8	12.9	7.5	43.8	
S-16	1100	19	16	19.5	7.3	40.4	
S-17	1600	10	10	14.5	5.4	37.4	
S-18	1200	5	4	18.9	6.9	40.7	
S-19	1700	3	3	10.0	6.6	36.1	
S-20	1200	5	4	10.6	7.5	59.3	
S-21	800	7.1	18	32.5	7.7	44.3	

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U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Soil Mechanics Laboratory

SOIL Soil Piping Study SOIL Nos. LOCATION

SOIL SURVEY LABORATORY Lincoln, Nebraska LAB. Nos. 71L118-71L129

GENERAL METHODS: 1B1b, 2A1, 2B

	SML No.	Extractable bases 5B4a 6N2e Ca	6O2d Mg	6P2a Na	6Q2a K	Sum	6H1a Ext. Acidity	Cat. Exch. Cap. 5A3a Sum Cations	5A6a NH_4OAc	Water extract from saturated paste 8A1 6N1b Ca	6O1b Mg	6P1b Na	6Q1b K		6I1a CO_3	6J1a HCO_3	6K1a Cl	6L1a SO_4	8A1a Electrical conductivity
		meq/100 g								meq/liter									mmho/cm
118	S-22	11.7	3.9	3.5	0.4	19.5			16.0	2.0	0.9	17.1	tr						2.14
119	S-23	1.7	5.2	1.1	0.3	8.3			17.5	tr	tr	1.0	tr						0.13
120	S-24	1.5	2.9	0.5	0.2	5.1			13.4	tr	tr	0.7	tr						0.10
121	S-25	0.7	4.1	0.4	0.2	5.4			16.0	tr	0.1	0.6	tr						0.08
122	S-26	41.9	11.8	4.8	0.4	58.4			29.5	0.3	0.2	6.9	tr						0.63
123	S-27	15.7	4.4	2.3	0.3	22.7			15.6	0.5	0.2	7.1	tr						0.77
124	S-28	10.6	3.5	3.5	0.3	17.9			11.0	0.8	0.4	11.7	0.1						1.42
125	S-29	8.9	5.5	6.4	0.4	21.2			15.5	1.6	1.5	33.8	0.1						3.77
126	S-30	19.8	6.1	3.6	0.3	29.8			15.7	0.3	0.2	9.9	tr						1.01
127	S-31	28.2	14.4	7.8	0.3	50.7			27.3	0.4	0.4	14.9	tr						1.68
128	S-32	50.1	19.6	0.2	0.4	70.3			36.3	2.3	1.6	0.4	tr						0.38
129	S-33	35.2	14.3	12.1	0.5	62.1			32.8	2.0	1.7	37.6	0.1						4.38

	SML No.	8E1 Resistivity ohms-cm	5D2 ESP Pct.	5E SAR	~~8D5~~ Total Sol. Salts me/1	8C1b pH Sat. Paste	8A Water at Sat. Pct.	
118	S-22	1000	17	14	20.0	7.4	43.8	
119	S-23	8000	6		1.0	4.8	40.5	
120	S-24	17000	4		0.7	5.0	37.9	
121	S-25	14000	3	3	0.7	4.8	41.5	
122	S-26	1400	15	14	7.4	7.9	56.9	
123	S-27	1800	13	12	7.8	7.9	38.7	
124	S-28	1200	28	15	13.0	8.0	35.5	
125	S-29	650	29	27	37.0	8.0	56.3	
126	S-30	1600	20	20	10.4	8.2	50.8	
127	S-31	960	25	24	15.7	8.1	57.2	
128	S-32	1400	1	<1	4.3	7.4	52.5	
129	S-33	470	29	28	41.4	7.8	68.9	

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U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL Soil Mechanics Laboratory
Soil Piping Study

SOIL Nos. ______ LOCATION ______

SOIL SURVEY LABORATORY Lincoln, Nebraska

LAB. Nos. 71L130-71L141

GENERAL METHODS: 1B1b, 2A1, 2B

	SML No.	Extractable bases 5B4a: 6N2e Ca	6O2d Mg	6P2a Na	6Q2a K	Sum	6H1a Ext. Acidity	Cat. Exch. Cap.: 5A3a Sum Cations	5A6a NH_4OAc	Water extract from saturated paste 8A1: 6N1b Ca	6O1b Mg	6P1b Na	6Q1b K		6I1a CO_3	6J1a HCO_3	6K1a Cl	6L1a SO_4	8A1a Electrical conductivity
		meq/100 g								meq/liter									mmho/cm
130	S-34	37.3	28.9	20.2	0.5	86.9			63.5	0.2	0.2	11.5	tr						
131	S-35	43.1	25.5	12.2	0.5	81.3			50.4	0.2	0.2	8.6	tr						1.01
132	S-36	43.4	2.9	26.5	0.6	73.4			54.3	0.2	0.2	21.8	tr						0.81
133	S-37	24.3	2.9	0.3	0.4	27.9			16.5	3.7	0.9	1.6	0.1						1.91
134	S-38	27.7	6.4	1.2	0.7	36.0			11.3	1.3	1.3	5.9	0.1						0.59
135	S-39	21.1	4.9	1.7	0.4	28.1			14.8	1.5	1.0	9.1	0.1						0.83
136	S-40	8.2	12.7	5.2	0.6	26.7			23.3	1.7	2.3	17.8	0.1						1.01
137	S-41	7.3	6.2	1.8	0.5	15.8			15.8	0.8	0.7	6.3	tr						2.35
138	S-42	4.2	3.2	0.6	0.2	8.2			12.8	0.1	0.1	1.0	tr						0.88
139	S-43	5.4	3.1	0.3	0.2	9.0			11.5	0.3	0.2	1.0	tr					0.20	~~[illegible]~~
140	S-44	2.4	3.1	1.4	0.2	7.1			13.0	0.3	0.3	4.7	tr					0.21	~~[illegible]~~
141	S-45	2.1	2.5	0.4	0.2	5.2			12.0	0.1	0.1	1.1	0.2						0.67
																			0.17

	SML No.	8E1 Resistivity ohms-cm	5D2 ESP Pct.	5E SAR	8D5 Total Sol. Salts me/l	8C1b pH Sat. Paste	8A Water at Sat. Pct.	
130	S-34	680	30	26	11.9	8.4	76.8	
	S-35	1000	23	19	9.0	8.1	78.7	
	S-36	600	43	49	22.2	8.5	157	
	S-37	2200	1	1	6.3	7.2	40.4	
	S-38	2100	9	5	8.6	7.9	37.8	
	S-39	1800	9	8	11.7	7.7	39.6	
	S-40	760	18	13	21.9	7.6	63.2	
	S-41	1800	9	7	7.8	7.2	47.7	
	S-42	9500	5	3	1.2	4.4	36.5	
	S-43	8300	3	2	1.5	5.3	38.4	
	S-44	4200	9	9	5.3	4.7	38.3	
	S-45	13000	3	3	1.5	5.0	36.2	

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U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL Soil Mechanics Laboratory
Soil Piping Study SOIL Nos. ________ LOCATION ________

SOIL SURVEY LABORATORY Lincoln, Nebraska LAB. Nos. 71L142-71L153

GENERAL METHODS: 1B1b, 2A1, 2B

	SML No.	Extractable bases 5B4a 6N2e Ca	6O2d Mg	6P2a Na	6Q2a K	Sum	6H1a Ext. Acidity	Cat. Exch. Cap. 5A3a Sum Cations	5A6a NH4OAc	Water extract from saturated paste 6N1b Ca	6O1b Mg	6P1b Na	6Q1b K		6I1a CO_3	6J1a HCO_3	6K1a Cl	6L1a SO_4	8A1a Electrical conductivity
		meq/100 g								meq/liter									mmho/cm
142	S-46	5.2	5.3	1.4	0.2	12.1			13.5	0.4	0.6	4.6	tr						0.67
143	S-47	4.7	5.6	1.6	0.2	12.1			12.7	0.5	0.9	6.3	tr						0.90
144	S-48	6.0	7.0	1.7	0.2	14.9			14.8	0.2	0.3	4.0	tr						0.55
145	S-49	4.0	3.2	0.7	0.2	8.1			10.6	0.4	0.4	2.7	tr						0.42
146	S-50	2.4	1.7	0.8	0.1	5.0			7.7	0.4	0.4	4.3	tr						0.63
147	S-51	13.8	8.9	4.7	0.3	27.7			21.0	15.0	14.5	25.3	0.1						4.59
148	S-52	10.2	7.0	2.8	0.2	20.2			14.2	2.1	2.1	13.7	tr						1.84
149	S-53	7.7	8.2	2.6	0.2	18.7			17.2	3.6	4.5	13.7	0.1						2.30
150	S-54	6.4	13.9	5.6	0.2	26.1			16.9	10.3	20.3	39.9	0.1						6.31
151	S-55	10.8	14.4	4.3	0.3	29.8			21.7	1.0	1.4	13.0	tr						1.66
152	S-56	11.4	13.9	5.5	0.2	31.0			20.1	14.0	17.0	32.1	0.1						5.61
153	S-57	3.2	3.7	0.8	0.1	7.8			9.2	1.0	1.2	4.8	0.1						0.81

	SML No.	8E1 Resistivity ohms-cm	5D2 ESP Pct.	5E SAR	8D5 Total Sol. Salts me/1	8C1b pH Sat. Paste	8A Water at Sat. Pct.	
142	S-46	2900	8	7	5.6	5.7	54.3	
	S-47	2400	10	8	7.7	6.2	53.5	
	S-48	3300	10	8	4.5	6.9	47.6	
	S-49	4700	6	4	3.5	5.1	44.2	
	S-50	4500	8	7	5.1	4.9	43.0	
	S-51	470	12	7	54.9	6.7	82.2	
	S-52	1300	14	9	17.9	7.9	55.4	
	S-53	1000	10	7	21.9	6.0	64.7	
	S-54	460	18	10	70.6	5.9	63.4	
	S-55	1200	15	12	15.4	7.3	74.3	
	S-56	460	15	8	63.1	6.1	74.8	
	S-57	3200	7	5	7.1	5.0	47.9	

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Soil Mechanics Laboratory
SOIL Soil Piping Study SOIL Nos. ______ LOCATION ______

SOIL SURVEY LABORATORY Lincoln, Nebraska LAB. Nos. 71L154-71L163, 71L169, 71L173
GENERAL METHODS: 1B1b, 2A1, 2B

	SML No.	Extractable bases 5B4a 6N2e Ca	6O2d Mg	6P2a Na	6Q2a K	Sum	6H1a Ext. Acidity	Cat. Exch. Cap. 5A3a Sum Cations	5A6a NH4OAc	Water extract from saturated paste 8A1 6N1b Ca	6O1b Mg	6P1b Na	6Q1b K		6I1a CO_3	6J1a HCO_3	6K1a Cl	6L1a SO_4	8A1a Electrical conductivity
		meq/100 g								meq/liter									mmho/cm
154	S-58	12.7	7.4	0.1	0.2	20.4			21.4	1.4	1.5	0.3	0.1						0.35
155	S-59	7.8	3.3	0.2	0.3	11.6			14.2	1.3	0.9	0.6	0.1						0.36
156	S-60	11.2	3.6	0.1	0.2	15.1			16.6	0.7	0.4	0.3	tr						0.18
157	S-72	16.4	10.8	1.1	0.5	28.8			25.0	5.0	3.6	5.2	0.2						1.36
158	S-73	17.0	10.8	0.9	0.6	29.3			24.6	10.5	8.0	5.4	0.4						2.30
159	S-61	11.7	6.5	0.5	0.4	19.1			20.7	0.6	0.6	0.8	tr						0.23
160	S-62	12.7	6.6	0.5	0.5	20.3			18.8	1.6	1.1	1.1	0.1						0.40
161	S-63	10.7	6.8	0.6	0.4	18.5			19.9	0.4	0.3	0.8	tr						0.19
162	S-64	10.7	6.5	0.5	0.5	18.2			19.4	0.6	0.4	0.7	tr						0.20
163	S-65	10.7	7.4	0.6	0.7	19.4			21.8	0.4	0.3	0.9	0.1						0.20
169	71F-22	4.5	3.7	1.2	0.2	9.6			10.1	0.2	0.2	3.4	0.2						0.49
173	S-67	6.3	0.4	0.1	0.3	7.1			2.1	21.8	3.5	1.2	0.5						2.22

	SML No.	8E1 Resis-tivity ohms-cm	5D2 ESP Pct.	5E SAR	8D5 Total Sol. Salts me/l	8C1b pH Sat. Paste	8A Water at Sat. Pct.	
154	S-58	5400	—	<1	3.3	6.5	33.2	
155	S-59	4500	1	1	2.9	6.1	40.0	
156	S-60	8500	1	<1	1.4	5.7	33.3	
157	S-72	900	4	3	14.0	7.1	43.7	
158	S-73	760	3	2	24.3	6.7	43.8	
159	S-61	3700	2	1	2.0	6.2	50.1	
160	S-62	2700	3	1	3.9	7.3	44.8	
161	S-63	3300	3	1	1.5	6.7	47.1	
162	S-64	3200	3	1	1.7	6.4	43.5	
163	S-65	2700	3	2	1.7	6.1	49.5	
169	71F-22	3300	11	8	4.0	5.3	39.4	
173	S-67	2100	5	<1	27.0	7.2	27.6	

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U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL Soil Mechanics Laboratory Soil Piping Study SOIL Nos. ______ LOCATION ______

SOIL SURVEY LABORATORY Lincoln, Nebraska LAB. Nos. 71L174 - 71L185

GENERAL METHODS: 1B1b, 2A1, 2B

	SML No.	Extractable bases 5B4a 6N2e Ca	6O2d Mg	6P2a Na	6Q2a K	Sum	6H1a Ext. Acidity	Cat. Exch. Cap. 5A3a Sum Cations	5A6a NH4OAc	Water extract from saturated paste 8A1 6N1b Ca	6O1b Mg	6P1b Na	6Q1b K		6I1a CO_3	6J1a HCO_3	6K1a Cl	6L1a SO_4	8A1a Electrical conductivity
		meq/100 g								meq/liter									mmho/cm
174	S-68.1	16.0	3.0	0.1	0.4	19.5			12.2	14.0	4.2	1.1	0.3						1.50
175	S-68.2	22.1	12.6	0.1	0.3	35.1			5.7	9.0	2.0	0.7	0.4						1.14
176	S-69	41.4	3.4	1.1	0.3	46.2			4.7	28.0	22.5	23.8	1.1						5.56
177	S-70	34.3	3.0	0.3	0.3	37.9			6.1	30.5	13.5	5.4	0.7						3.49
178	S-71	35.3	3.6	0.3	0.3	39.5			5.9	19.3	11.0	5.1	0.7						2.72
179	S-76.1	4.4	1.7	0.1	0.2	6.4			8.5	0.4	0.2	0.6	tr						0.17
180	S-76.2	4.0	1.7	0.1	0.2	6.0			7.8	0.6	0.3	0.5	0.1						0.18
181	S-77	4.1	4.1	2.9	0.2	11.3			11.9	0.7	0.9	14.1	0.1						1.72
182	S-78.1	4.9	4.1	0.6	0.3	9.9			13.4	0.2	0.2	1.5	0.1						0.25
183	S-78.2	5.6	5.1	1.7	0.3	12.7			14.5	0.4	0.5	5.8	0.1						0.78
184	S-79	5.4	3.7	0.8	0.3	10.2			11.9	0.2	0.2	2.1	tr						0.26
185	S-80	12.7	5.6	0.1	0.4	18.8			14.0	3.0	2.0	0.6	0.1						0.57

	SML No.	8E1 Resistivity ohms-cm	5D2 ESP Pct.	5E SAR	8D5 Total Sol. Salts me/l	8C1b pH Sat. Paste	8A Water at Sat. Pct.	
174	S-68.1	1400	<1	<1	19.6	7.5	44.5	
	S-68.2	2000	2	<1	12.1	7.5	37.7	
	S-69	840	9	5	75.4	7.6	27.6	
	S-70	1000	3	1	50.1	7.6	26.8	
	S-71	1300	3	1	36.1	7.7	27.8	
	S-76.1	9000	1	1	1.2	5.2	35.0	
	S-76.2	9500	1	1	1.5	5.3	32.9	
Ark	S-77	1500	19	16	15.8	5.3	39.2	
	S-78.1	5100	4	3	2.0	4.6	37.9	
	S-78.2	2300	10	9	6.8	5.0	42.2	
	S-79	4900	6	5	2.5	5.3	33.4	
	S-80	2600	1	<1	5.7	7.6	42.9	

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U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Soil Mechanics Laboratory

SOIL Soil Piping Study SOIL Nos. ________ LOCATION ________

SOIL SURVEY LABORATORY Lincoln, Nebraska LAB. Nos. 71L186 - 71L197

GENERAL METHODS: 1B1b, 2A1, 2B

	SML No.	Extractable bases 5B4a 6N2e Ca	6O2d Mg	6P2a Na	6Q2a K	Sum	6H1a Ext. Acidity	Cat. Exch. Cap. 5A3a Sum Cations	5A6a NH_4OAc	Water extract from saturated paste 6N1b Ca	6O1b Mg	6P1b Na	6Q1b K		6I1a CO_3	6J1a HCO_3	8A1 6K1a Cl	6L1a SO_4	8A1a Electrical conductivity
		meq/100 g								meq/liter									mmhos/cm
186	S-81	9.7	6.3	0.2	0.3	16.5			15.9	2.5	1.6	0.9	tr						0.51
187	S-82	11.2	7.0	0.4	0.3	18.9			16.3	14.0	11.0	3.5	0.1						2.17
188	S-83	9.4	3.4	0.1	0.4	13.3			10.9	4.4	2.0	0.6	0.3						0.70
189	S-84	8.7	7.9	0.2	0.6	17.4			23.9	0.8	0.8	0.5	0.2						0.28
190	S-85	6.0	3.6	0.6	0.2	10.4			11.1	0.2	0.1	1.5	tr						0.23
191	S-86	4.4	2.5	0.7	0.2	7.8			9.1	0.1	0.1	1.4	tr						0.19
192	S-87	5.1	3.8	1.7	0.2	10.8			10.2	1.1	0.9	10.9	0.1						1.40
193	S-88	5.4	4.3	0.9	0.2	10.8			11.3	0.1	0.1	2.1	tr						0.26
194	S-89	5.1	4.2	0.9	0.2	10.4			11.5	0.2	1.5	2.3	tr						0.32
195	S-90	6.1	4.4	0.3	0.2	11.0			9.7	0.9	0.9	1.5	tr						0.34
196	S-91	5.6	4.2	0.2	0.2	10.2			10.3	0.7	0.7	0.7	tr						0.23
197	S-92	8.7	6.3	1.6	0.4	17.0			15.4	2.6	2.8	10.3	0.1						1.57

	SML No.	8E1 Resistivity ohms-cm	5D2 ESP Pct.	5E SAR	8D5 Total Sol. Salts me/1	8C1b pH Sat. Paste	8A Water at Sat. Pct.
186	S-81	2300	1	1	5.0	7.5	42.2
	S-82	1100	1	1	28.6	6.7	42.2
	S-83	2700	1	<1	7.3	7.1	35.3
	S-84	3000	1	1	2.3	4.3	52.1
	S-85	5200	5	4	1.8	5.8	49.2
	S-86	6800	7	4	1.6	5.6	47.0
	S-87	1700	13	11	13.0	6.4	34.6
	S-88	3800	7	7	2.3	5.1	38.1
	S-89	3900	7	2	4.0	4.9	43.5
	S-90	4300	2	2	3.3	7.3	37.0
	S-91	4600	2	1	2.1	6.5	42.1
	S-92	1500	7	6	15.8	7.2	46.7

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U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL Soil Mechanics Laboratory
Soil Piping Study
SOIL Nos. LOCATION

SOIL SURVEY LABORATORY Lincoln, Nebraska LAB. Nos. 71L198-71L209

GENERAL METHODS: 1B1b, 2A1, 2B

	SML No.	Extractable bases 5B4a: 6N2e Ca	6O2d Mg	6P2a Na	6Q2a K	Sum	6H1a Ext. Acidity	Cat. Exch. Cap. 5A3a Sum Cations	5A6a NH4OAc	Water extract from saturated paste 8A1: 6N1b Ca	6O1b Mg	6P1b Na	6Q1b K		6I1a CO_3	6J1a HCO_3	6K1a Cl	6L1a SO_4	8A1a Electrical conductivity
		meq/100 g								meq/liter									mmho/cm
198	S-93	13.7	9.7	1.6	0.6	25.6			22.9	1.0	1.1	4.6	0.1						0.75
199	S-94	15.2	8.9	2.1	0.3	26.5			20.7	6.0	5.5	11.1	0.1						2.04
200	S-95	30.1	6.6	3.0	1.4	41.1			26.6	34.3	9.8	25.3	0.5						6.81
201	S-96	32.6	6.3	4.0	1.1	44.0			29.0	22.8	6.5	33.8	0.3						5.68
202	S-97	8.4	5.1	0.5	0.4	14.4			14.3	0.4	0.3	1.4	0.1						0.26
203	S-99	34.1	6.2	10.3	1.0	51.6			36.7	29.0	6.0	41.5	0.5						7.23
204	S-201a									0.1	0.2	0.2	0.1		–	–	0.1		0.10
205	S-202a									0.1	0.2	0.1	0.1		–	–	0.5		0.08
206	S-203a									0.4	0.4	1.0	0.1		–	1.0	0.7		0.30
207	S-204a									0.1	0.2	0.2	0.1		–	0.8	–		0.11
208	S-205a									0.4	0.4	0.3	0.1		–	1.5	–		0.20
209	S-206a									0.1	0.3	0.3	0.1		–	1.0	–		0.15

	SML No.	8E1 Resistivity ohms-cm	5D2 ESP Pct.	5E SAR	8D5 Total Sol. Salts me/l	8C1b pH Sat. Paste	8A Water at Sat. Pct.	
198	S-93	1100	6	4	6.8	6.9	55.6	
199	S-94	2000	8	5	22.7	7.6	41.5	
200	S-95	380	6	5	69.9	7.4	51.8	
201	S-96	430	8	9	63.4	7.3	51.8	
202	S-97	3800	3	2	2.2	5.5	28.9	
203	S-99	260	18	10	77.0	7.0	92.5	
204	S-201a			1	0.6	6.1		
205	S-202a			<1	0.5	6.7		
206	S-203a			2	1.9	7.2		
207	S-204a			1	0.6	6.9		
208	S-205a			<1	1.2	7.2		
209	S-206a			1	0.8	7.0		

a. WATER SAMPLES

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U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Soil Mechanics Laboratory

SOIL Soil Piping Study SOIL Nos. ______ LOCATION ______

SOIL SURVEY LABORATORY Lincoln, Nebraska LAB. Nos. 71L210-71L219

GENERAL METHODS: 1B1b, 2A1, 2B

	SML No.	Extractable bases 5B4a 6N2e Ca	6O2d Mg	6P2a Na	6Q2a K	Sum	6H1a Ext. Acidity	Cat. Exch. Cap. 5A3a Sum Cations	5A6a NH4OAc	Water extract from saturated paste 8A1 6N1b Ca	6O1b Mg	6P1b Na	6Q1b K		6I1a CO_3	6J1a HCO_3	6K1a Cl	6L1a SO_4	8A1a Electrical conductivity
		meq/100 g								meq/liter									mmho/cm
210	5-207a									0.1	0.2	0.2	0.1		—	0.5	—		0.10
211	5-208a									0.1	0.3	0.3	0.2		—	0.8	—		0.14
212	5-209a									0.3	0.5	0.6	0.1		—	2.3	—		0.24
213	5-210a									0.5	0.7	1.1	0.1		—	3.3	—		0.39
214	5-211a									0.4	0.7	1.3	0.1		—	2.3	—		0.41
215	5-212a									0.2	0.6	0.6	0.1		—	2.3	0.2		0.24
216	5-213a									tr	0.1	0.2	tr		—	0.3	—		0.08
217	5-214a									0.1	0.1	0.1	0.1		—	0.8	—		0.08
218	5-215a									0.4	1.5	0.5	tr		—	3.8	—		0.37
219	5-216a									0.1	0.1	0.1	tr		—	0.8	—		0.06

	SML No.	8E1 Resistivity ohms-cm	5D2 ESP Pct.	5E SAR	8D5 Total Sol. Salts me/l	8C1b pH Sat. Paste	8A Water at Sat. Pct.	
210	5-207a			1	0.6	6.7		
211	5-208a			1	0.9	7.0		
212	5-209a			1	1.5	7.5		
213	5-210a			1	2.4	7.5		
214	5-211a			2	2.5	7.5		
215	5-212a			1	1.5	7.3		
216	5-213a			1	0.3	6.7		
217	5-214a			<1	0.4	6.6		
218	5-215a			1	2.4	8.1		
219	5-216a			<1	0.3	7.3		

a. Water Sample

APPENDIX E

RAPID DISPERSION (CRUMB) TEST

RAPID DISPERSION (CRUMB) TEST

As discussed in Appendix A, several Australian investigators have concluded that a rapid and simple dispersion test may give as good a measure of susceptibility to piping as the more elaborate chemical tests (see particularly Emerson, 1964 and 1967, and Rallings, 1966).

The test consists of dropping an air-dried soil crumb (about 1 to 2 grams) into a small beaker (100 to 150 milliliters) of pure water and observing the tendency for the water adjacent to the crumb to become colored by clay particles in suspension.

All the soil samples gathered in this study were tested in this fashion. Four categories of "reaction" were defined and employed to judge the test result as given in Table E-1. In addition to pure (demineralized) water, tests were made in dilute sodium hydroxide solutions (0.001 and 0.006 Normal). Three individuals examined and rated the results of the tests independently: there was remarkably little disagreement among the three ratings. The average results are given in Table E-2.

In order to study the influence of the original water content of the crumbs, seven of the soils were chosen (randomly to represent both dispersive and non-dispersive soils), and tests were performed on ove-dried samples and also on samples which had been wetted and remolded to a water content midway between the liquid and plastic limits. The results, which are tabulated below, show that the water content of the crumb were not very important. The oven-dried specimens slaked and crumbled more rapidly and actively than the wetter specimens but the colloidal cloud reaction was not greatly different.

	Reaction to Rapid Dispersion (Crumb) Test	
Sample	Oven Dry	Wet
S-10	1	2
S-20	1	1
S-30	4	4
S-33	2	4
S-43	1	2
S-47	2	3
S-95	1	1
S-99	1	1

Table E-1

Reaction to Crumb Test

Grade	Definition
(1)	No Reaction: Crumb may slake and run out on bottom of beaker in flat pile but no sign of cloudy water caused by colloids in suspension.
(2)	Slight Reaction: Bare hint of cloud in water at the surface of crumb. (If the cloud is easily visible, use Group 3)
(3)	Moderate Reaction: Easily recognizable cloud of colloids in suspension. Usually spreading out in thin streaks on bottom of beaker. Doesn't cover whole bottom area.
(4)	Strong Reaction: Colloidal cloud covers nearly whole bottom of beaker, usually in a very thin skin. In extreme cases all the water in the beaker becomes cloudy.

Table E-2

RAPID DISPERSION (CRUMB) TEST RESULTS

Sample No.	Reaction* A	Reaction* B	Reaction* C	Sample No.	Reaction* A	Reaction* B	Reaction* C	Sample No.	Reaction* A	Reaction* B	Reaction* C
S-1	2	4	4	S-17	2	3	4	S-34	2	3	2
2	1	3	4	18	1	2	2	35	2	2	1
3	3	4	4	19	1	2	2	36	4	4	3
4	1	2	4	20	1	2	2	37	1	1	3
5	3	3	4	21	4	4	4	38	3	3	4
6	3	3	4	22	4	4	4	39	2	1	3
7.1	1	2	4	23	1	2	4	40	4	4	4
7.2	2	3	4	24	1	2	4	41	4	4	4
8	4	4	3	25	2	4	4	42	3	2	4
9	1	1	2	26	1	1	1	43	4	4	3
10	1	1	2	27	3	2	3	44	3	2	4
11	4	4	4	28	3	4	4	45	2	3	4
12	4	4	4	29	3	3	3	46	3	3	4
13	3	2	3	30	4	4	4	47	4	3	4
14	1	2	4	31	2	1	3	48	3	3	4
15	1	1	4	32	1	1	1	49	1	2	4
16	4	4	4	33	3	4	3	50	1	2	4

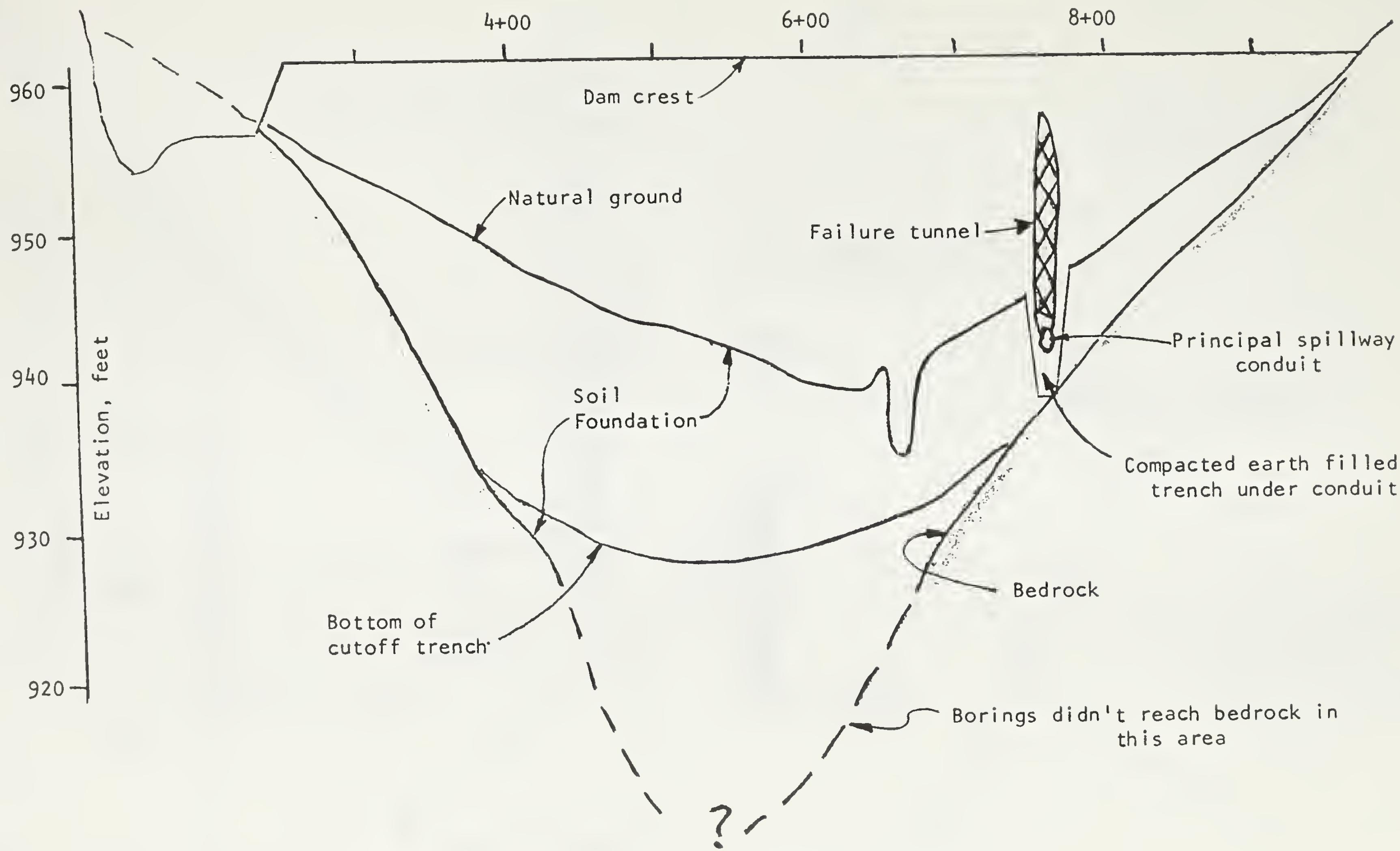
*Symbol	Solution	pH	Resistance (ohms/cm)
A	Demineralized H_2O	4.5	75,000
B	0.001 Normal NaOH	8.5	11,000
C	0.006 " "	10.7	1,100

Table E-2

Sample No	Reaction A	Reaction B	Reaction C	Sample No.	Reaction A	Reaction B	Reaction C	Sample No.	Reaction A	Reaction B	Reaction C
S-51	2	2	3	S-69	2	2	2	S-85	1	1	4
52	2	2	2	70	2	2	2	86	1	2	3
53	1	2	4	71	2	2	2	87	3	2	4
54	1	1	4	72	2	1	2	88	3	3	4
55	2	2	3	73	1	1	2	89	4	3	4
56	2	2	3	74	1	1	2	90	3	2	2
57	1	2	4	75	1	2	4	91	2	2	3
58	1	1	1	76.1	2	2	3	92	1	1	1
59	1	2	2	76.2	2	2	3	93	1	1	1
60	1	2	3	77	4	3	4	94	1	1	2
61	1	1	2	78.1	1	2	4	95	1	1	1
62	1	1	3	78.2	3	3	4	96	1	1	1
63	1	1	3	79	2	2	4	97	1	1	2
64	1	1	3	80	1	1	2	98	1	1	2
66	3	2	4	81	1	1	2	99	1	1	1
67	2	3	3	82	1	1	2				
68.1	1	1	4	83	1	1	3				
68.2	1	1	2	84	1	2	4				

APPENDIX F

OKLAHOMA DAMS - LONGITUDINAL SECTIONS
SHOWING LOCATION OF PIPING FAILURE
(TUNNEL OR BREACH) RELATIVE TO SLOPE
OF BEDROCK SURFACE AND SPILLWAY CONDUITS.

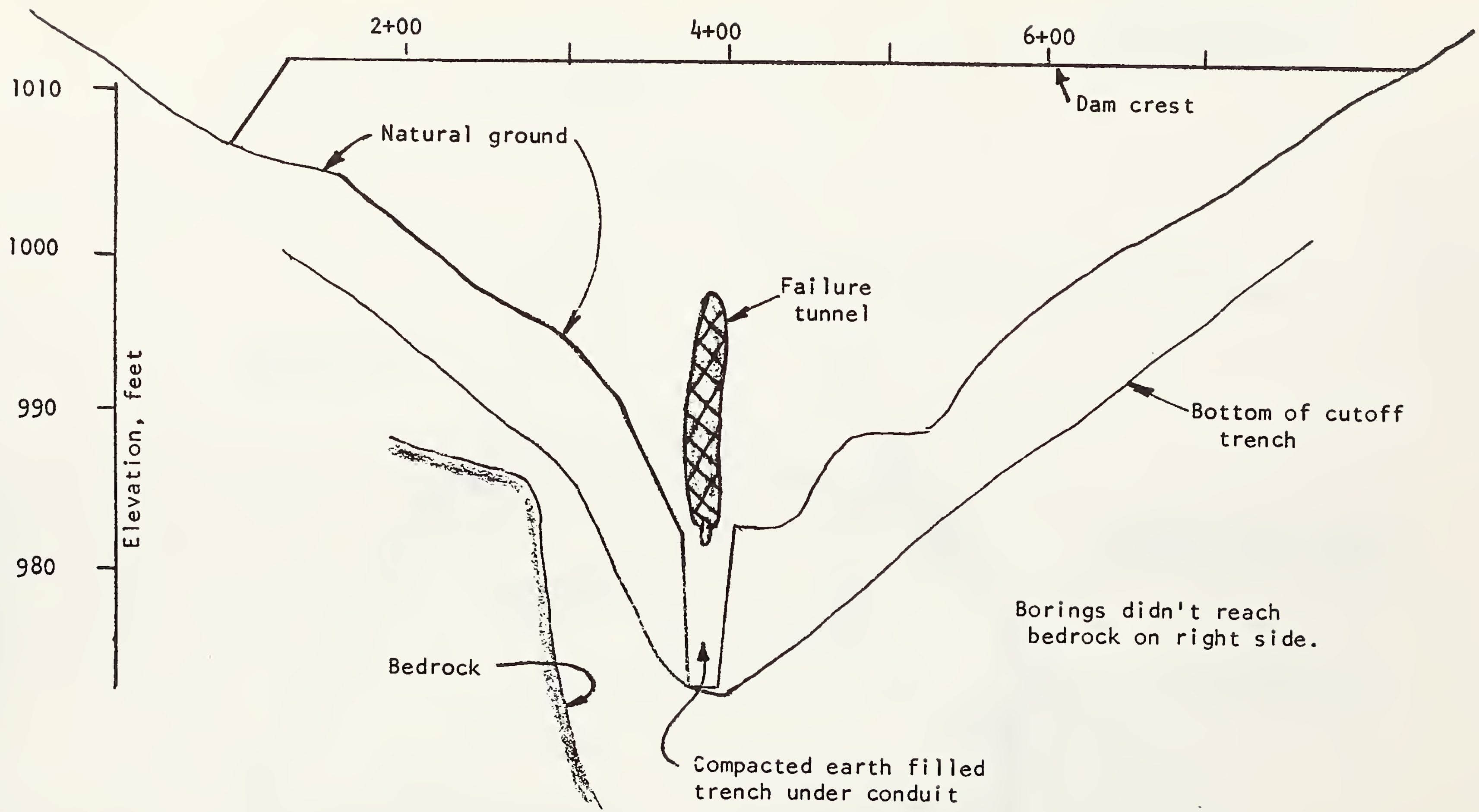


Scale: 1 inch = 10 ft. vertical
1 inch = 100 ft. horizontal

Owl Creek Site 7
(Longitudinal section looking downstream)

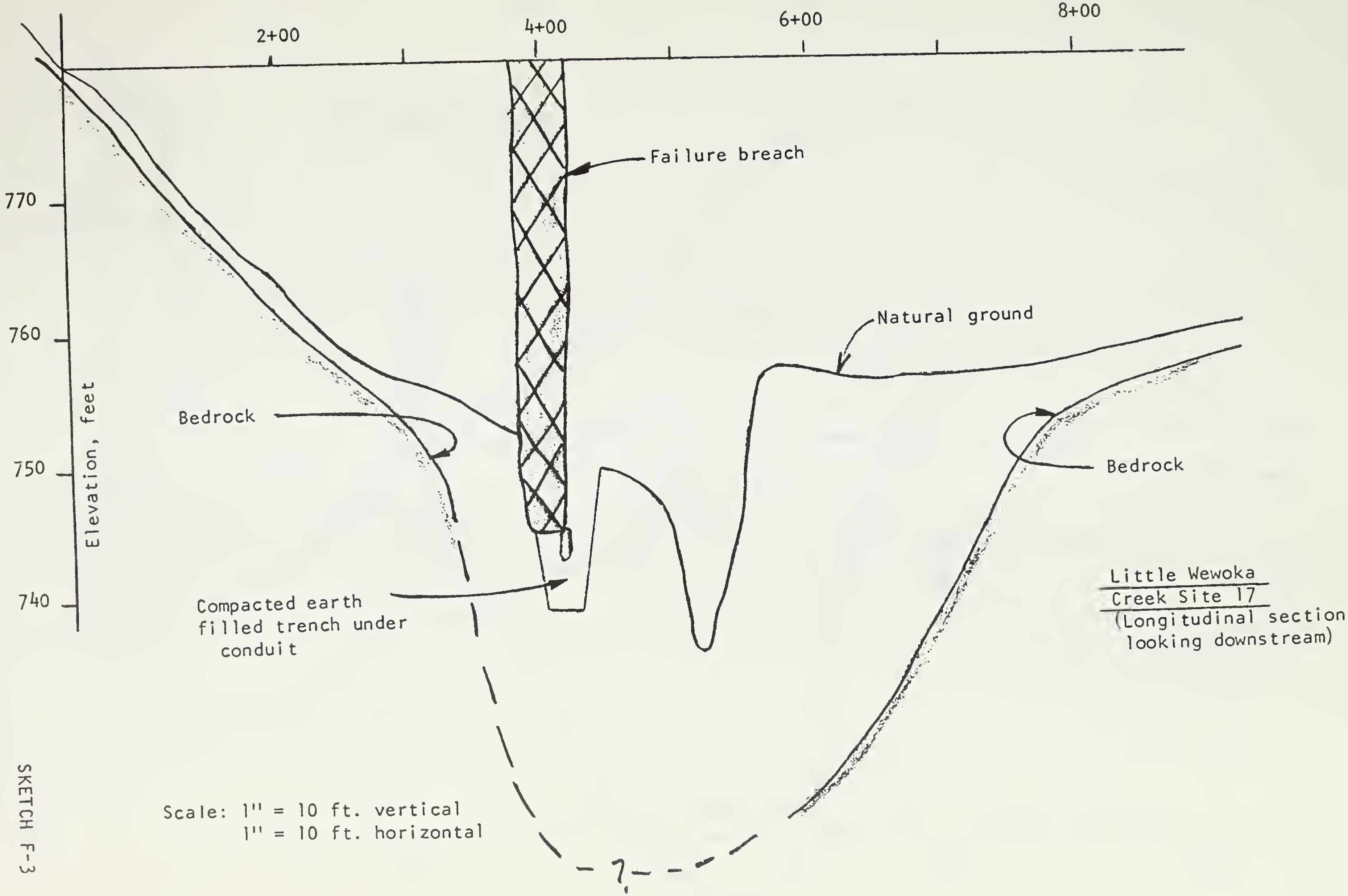
SKETCH F-1

SKETCH F-2

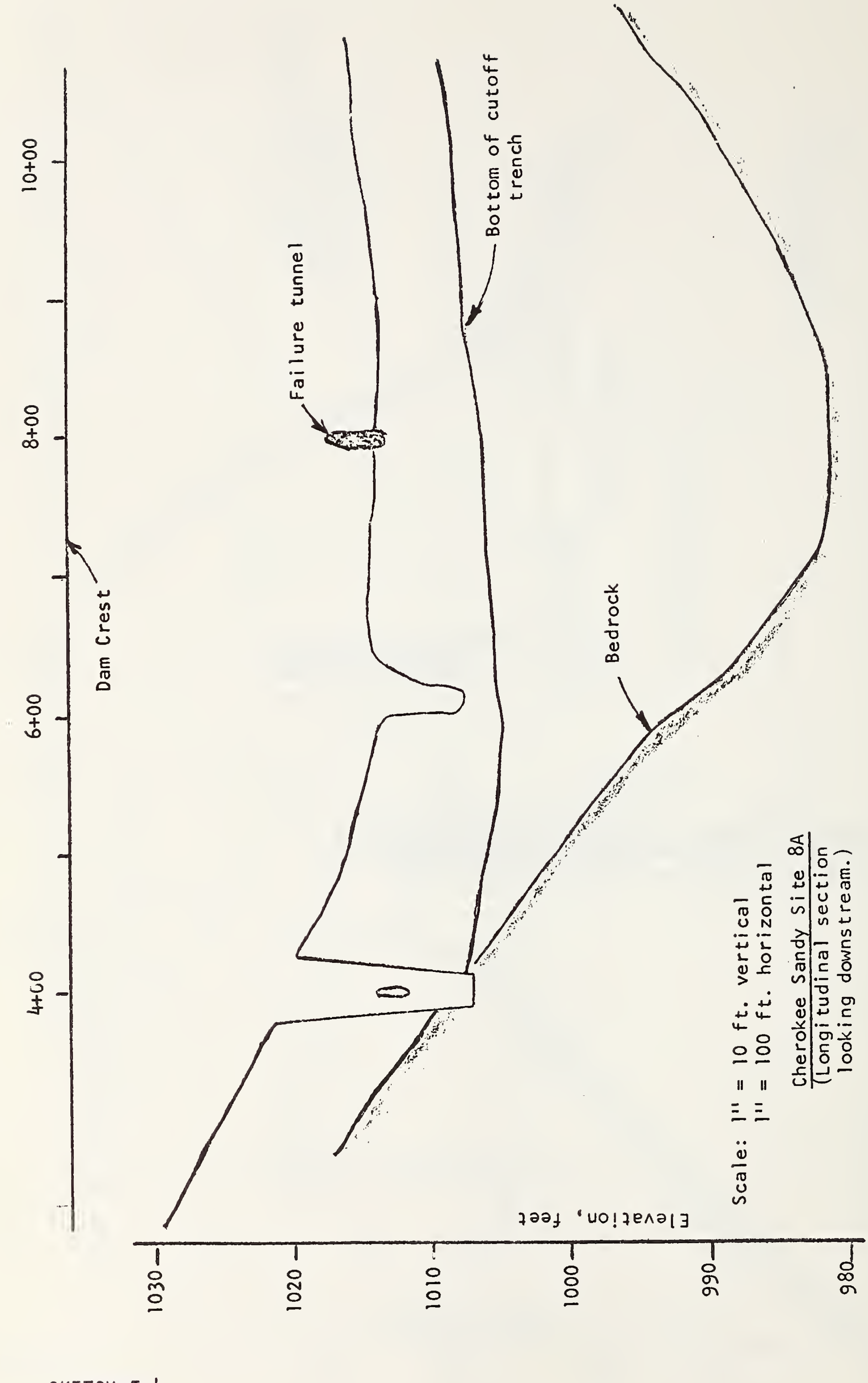


Scale: 1" = 10 ft. vertical
1" = 100 ft. horizontal

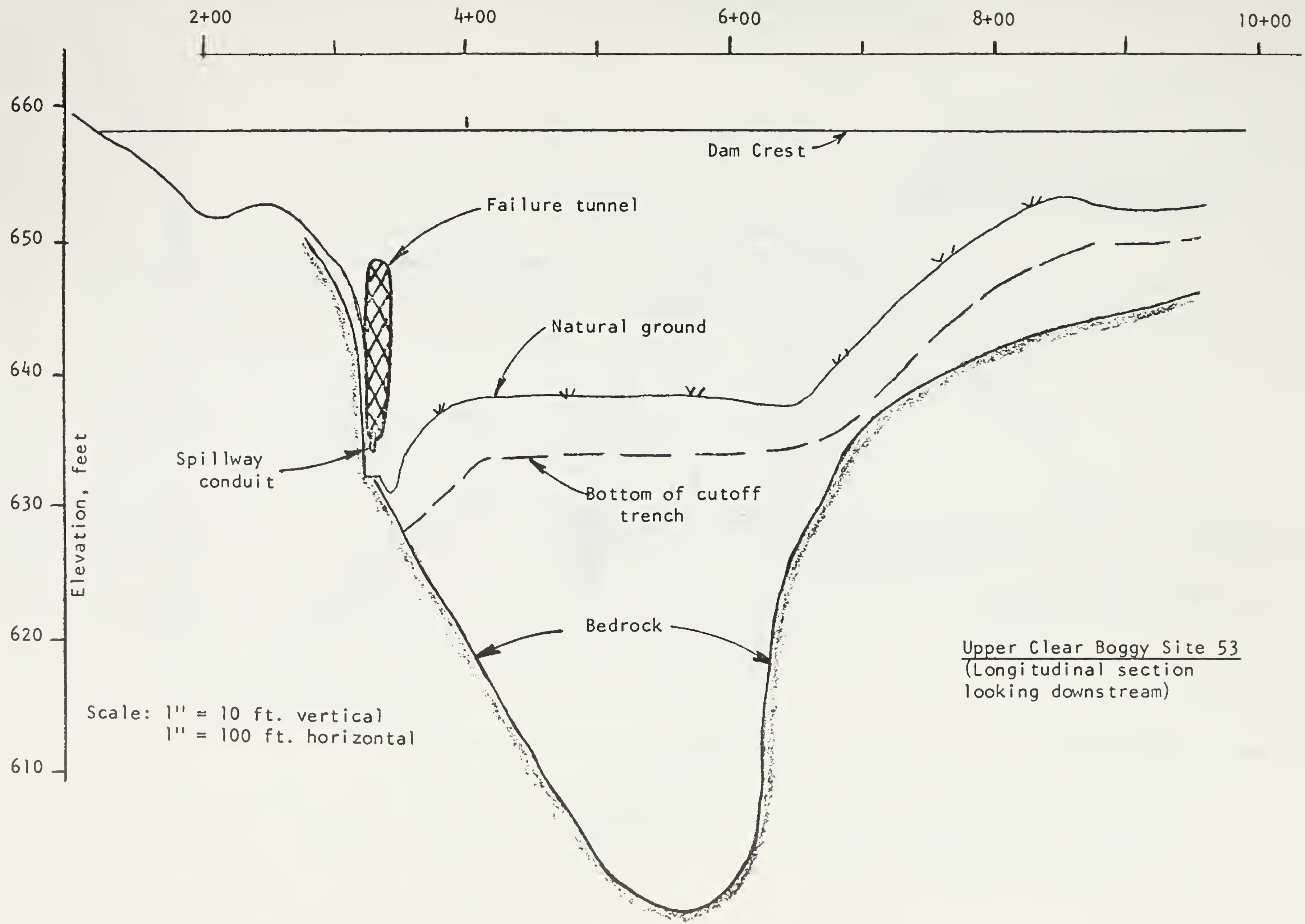
Owl Creek Site 13
(Longitudinal section looking downstream)


2+00
4+00
6+00
8+00
770
760
750
740
Elevation, feet
Failure breach
Natural ground
Bedrock
Bedrock
Compacted earth filled trench under conduit
?
Little Wewoka Creek Site 17
(Longitudinal section looking downstream)
Scale: 1" = 10 ft. vertical
1" = 10 ft. horizontal

SKETCH F-3


10+00
8+00
6+00
4+00
Dam Crest
Failure tunnel
Bottom of cutoff trench
Bedrock
Elevation, feet
1030
1020
1010
1000
990
980
Scale: 1" = 10 ft. vertical
1" = 100 ft. horizontal
Cherokee Sandy Site 8A
(Longitudinal section looking downstream.)

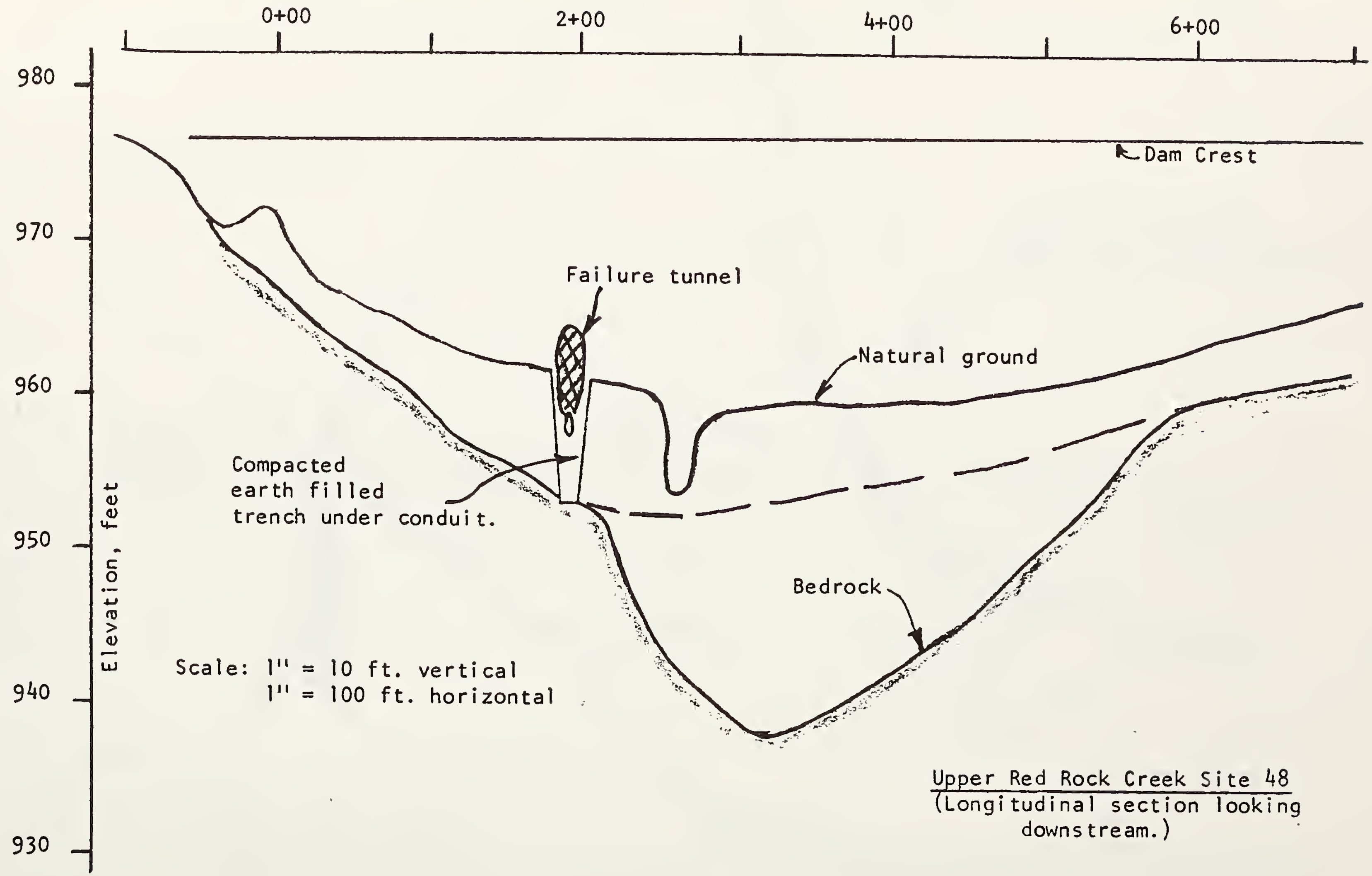
SKETCH F-4



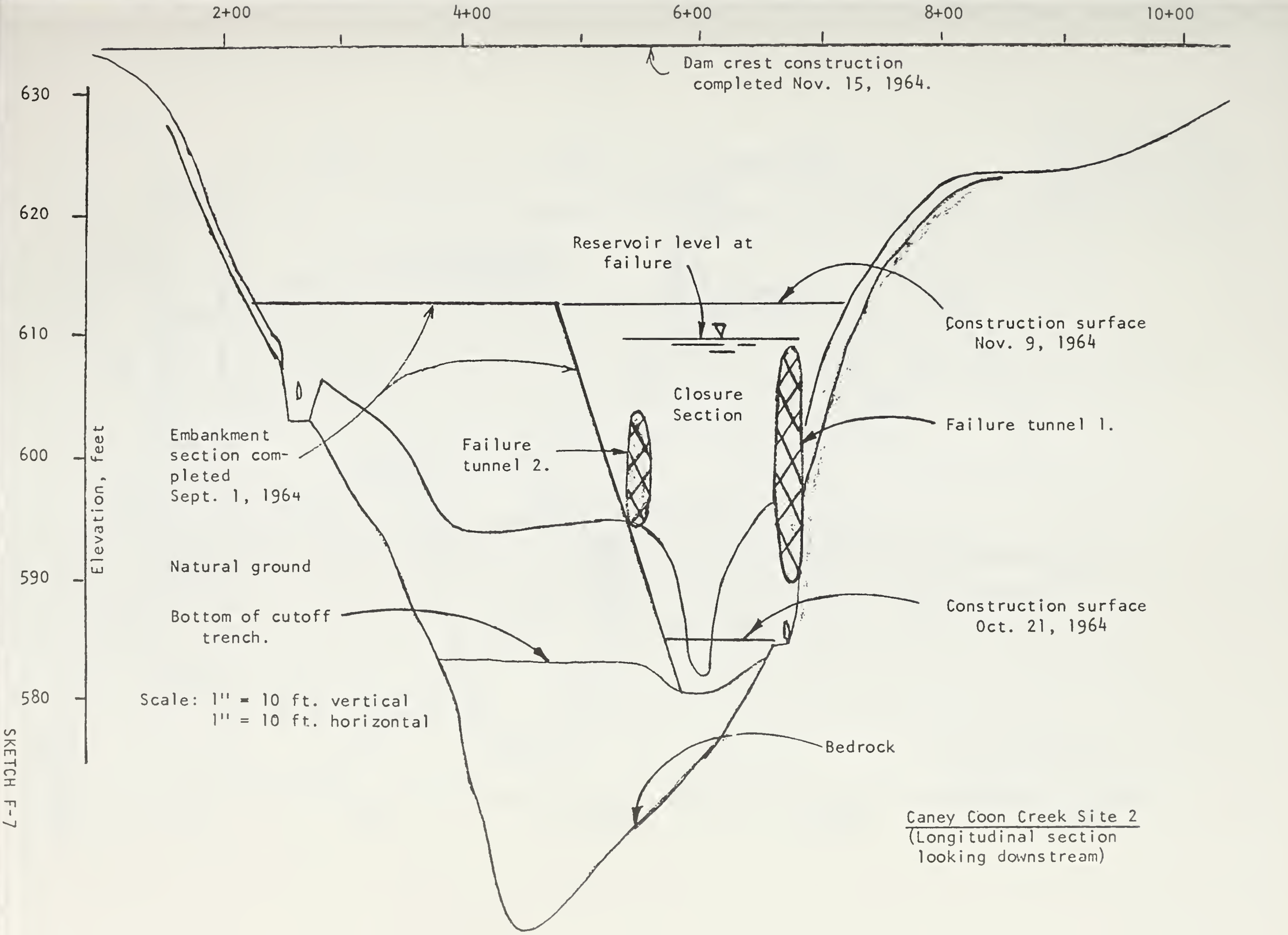
2+00
4+00
6+00
8+00
10+00
660
650
640
630
620
610
Elevation, feet
Dam Crest
Failure tunnel
Natural ground
Spillway conduit
Bottom of cutoff trench
Bedrock
Scale: 1" = 10 ft. vertical
1" = 100 ft. horizontal
Upper Clear Boggy Site 53
(Longitudinal section looking downstream)

SKETCH F-5

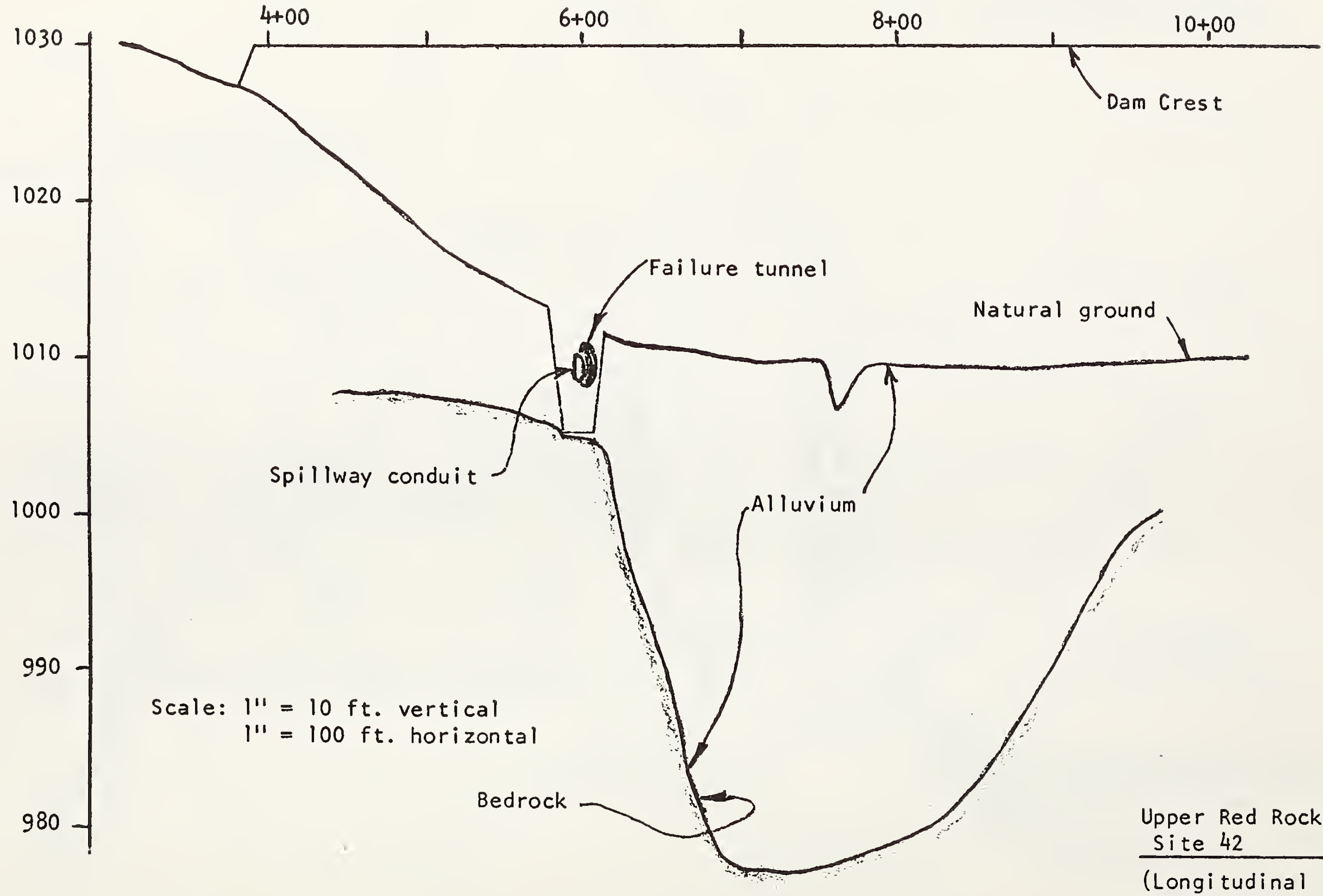
SKETCH F-6



Upper Red Rock Creek Site 48
(Longitudinal section looking downstream.)


2+00
4+00
6+00
8+00
10+00
Dam crest construction
completed Nov. 15, 1964.
630
620
610
600
590
580
Elevation, feet
Reservoir level at
failure
Construction surface
Nov. 9, 1964
Closure
Section
Failure tunnel 1.
Failure
tunnel 2.
Embankment
section com-
pleted
Sept. 1, 1964
Natural ground
Bottom of cutoff
trench.
Construction surface
Oct. 21, 1964
Scale: 1" = 10 ft. vertical
1" = 10 ft. horizontal
Bedrock
Caney Coon Creek Site 2
(Longitudinal section
looking downstream)

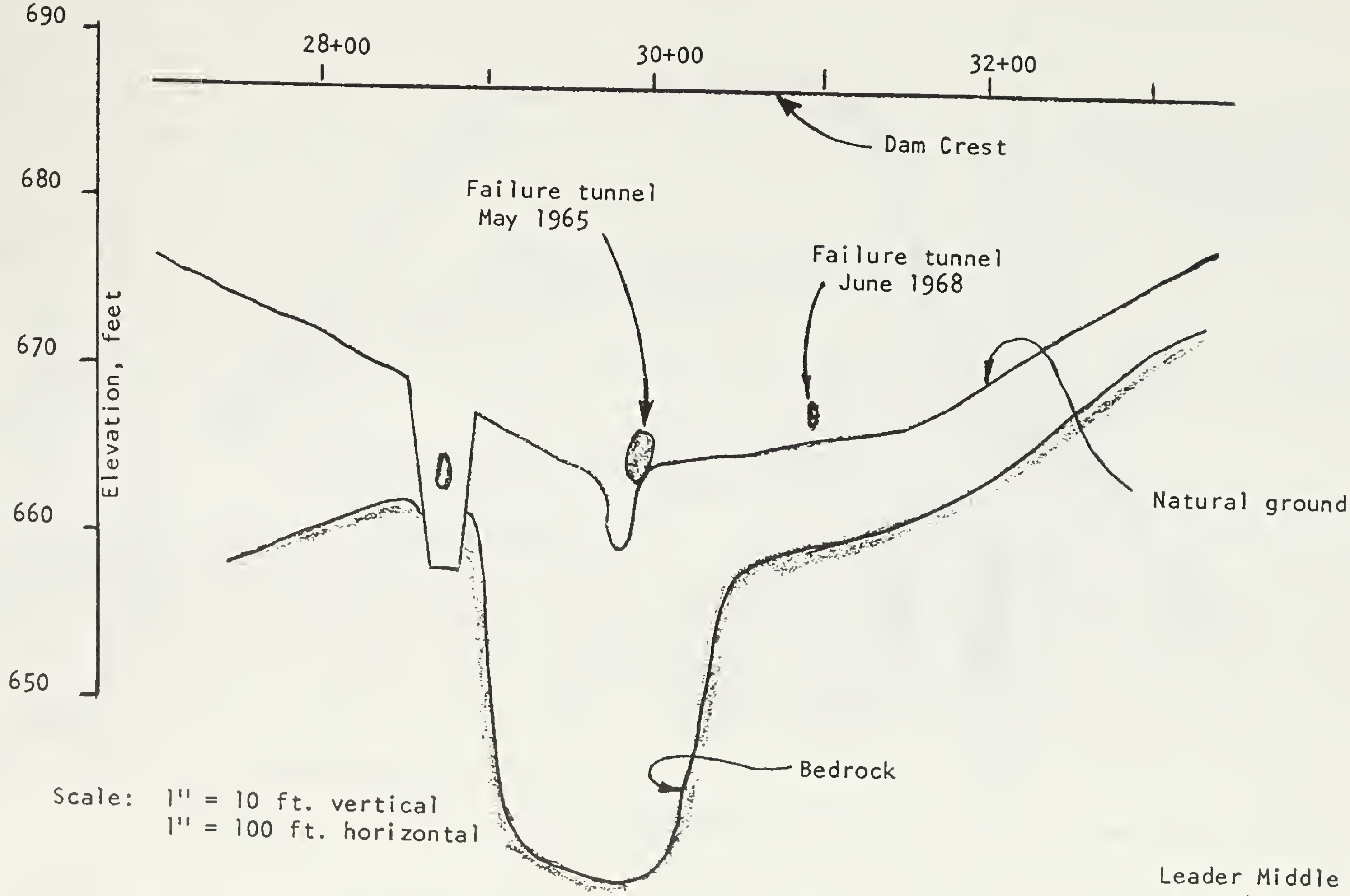
SKETCH F-7



Upper Red Rock Creek
Site 42

(Longitudinal section looking downstream)

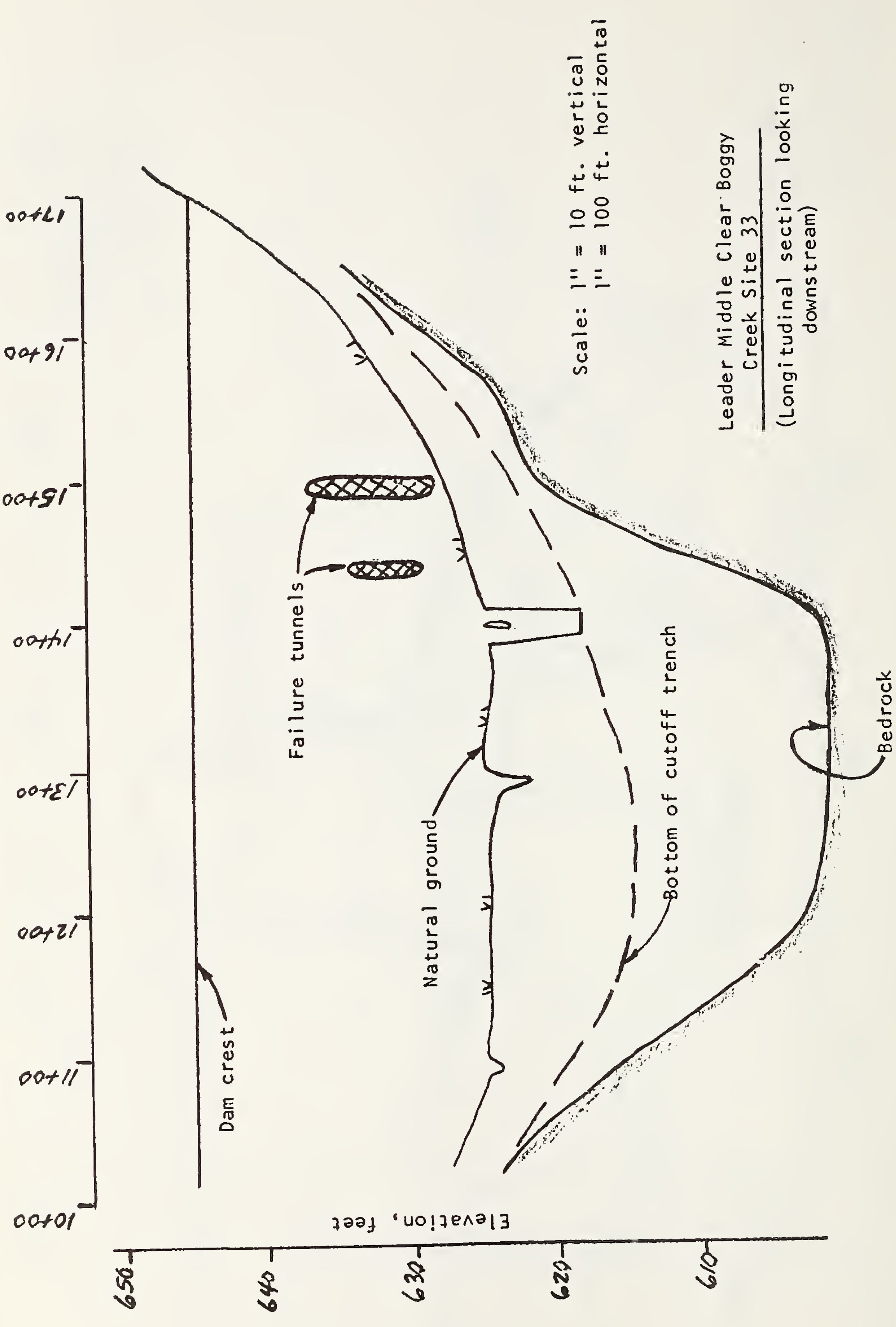
SKETCH F-8



Leader Middle Boggy Creek
Site 15

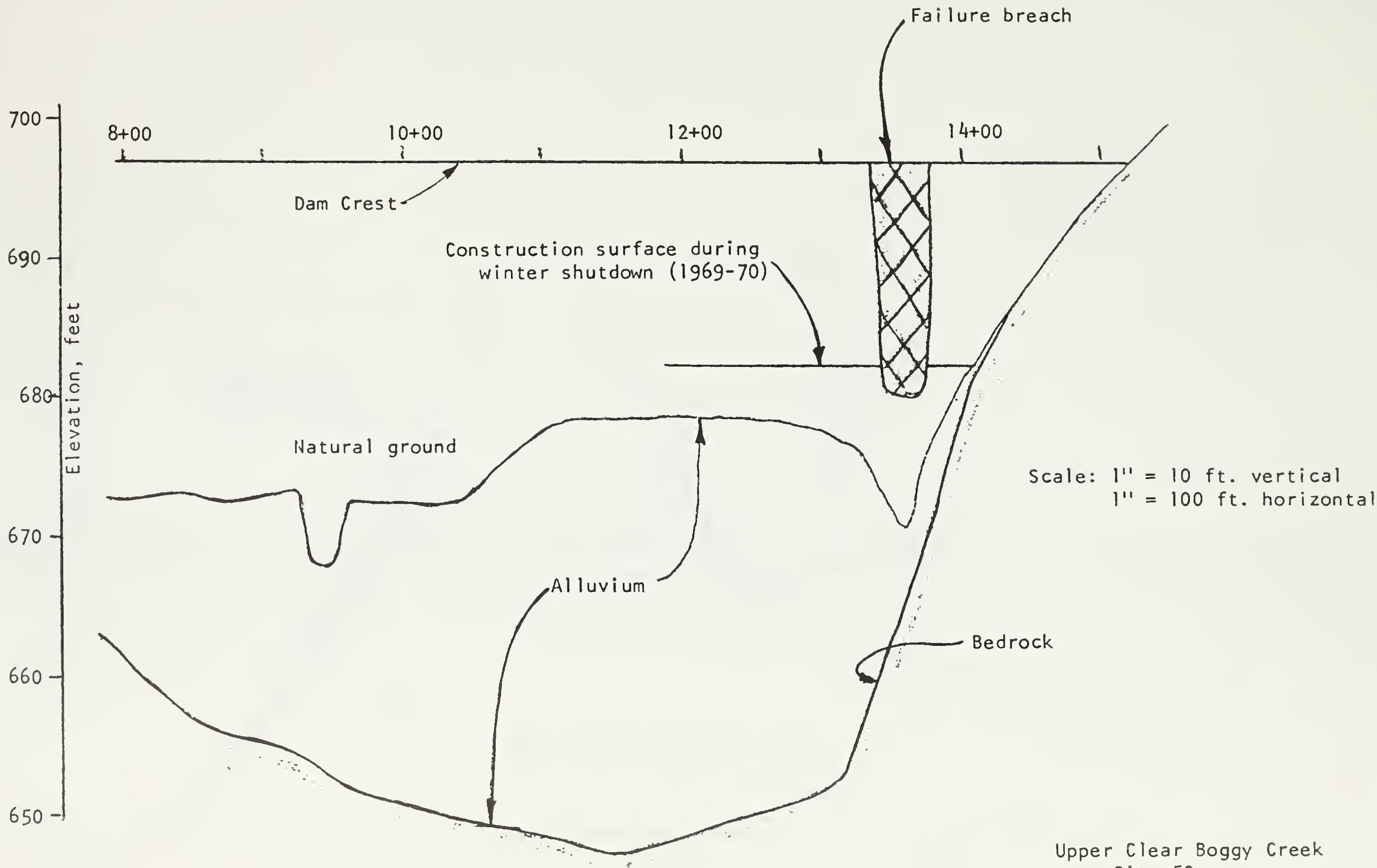
(Longitudinal Section
looking downstream)

SKETCH F-9



Scale: 1" = 10 ft. vertical
1" = 100 ft. horizontal
Leader Middle Clear Boggy Creek Site 33
(Longitudinal section looking downstream)
Failure tunnels
Bottom of cutoff trench
Bedrock
Natural ground
Dam crest
Elevation, feet
650
640
630
620
610
10+00
11+00
12+00
13+00
14+00
15+00
16+00
17+00

SKETCH F-10

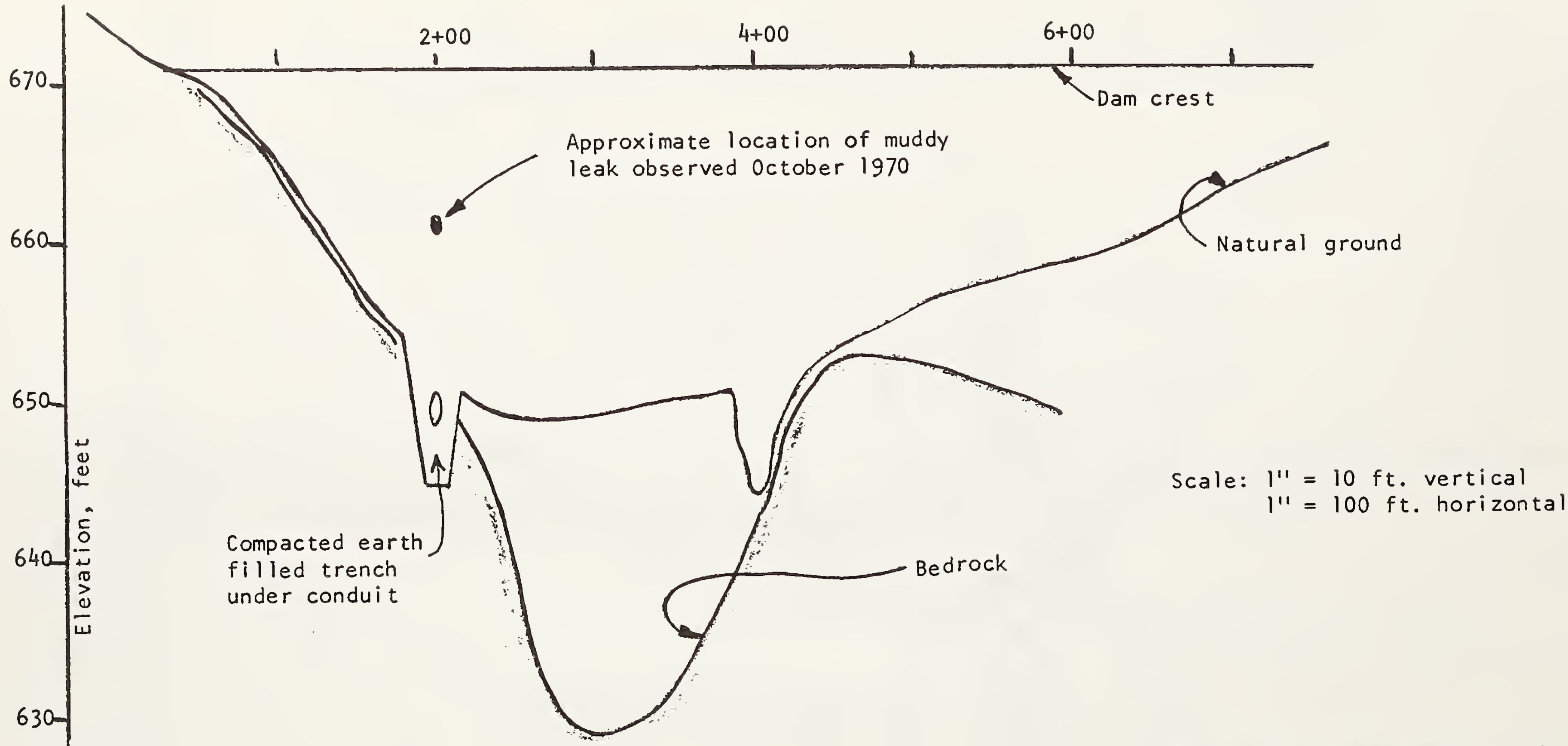


Upper Clear Boggy Creek
Site 50

(Longitudinal section
looking downstream)

SKETCH F-11

SKETCH F-12



Leader Middle Clear Boggy Creek
Site 29

(Longitudinal section looking downstream)

APPENDIX G

STRESS CALCULATIONS
FINITE ELEMENT ANALYSES

STRESS CALCULATIONS -- FINITE ELEMENT ANALYSES

During the spring of 1971, while the writer was engaged in this investigation, Professor J. M. Duncan and Dr. G. LeFebvre, Department of Civil Engineering, University of California, were carrying out continuing developmental work on the application of the finite element method to the calculation of stresses and strains in embankments. At the writer's suggestion, they included in their study an example with dimensions and details approximating the average conditions for the SCS Oklahoma dams[1] -- that is: (1) a dam of moderately stiff clay of about 40 feet in height; (2) incompressible rock abutment; (3) soil layers in foundation causing a settlement of the order of 12 to 18 inches, which may occur wholly during construction or partially during and after construction.

Computations were made of the stress changes in the portion of the embankment near the rock abutment as a function of varying amounts and times of foundation settlement. The problem analyzed is shown in Sketch G-1. Various amounts of foundation settlement were introduced by varying the compressibility of the "soil foundation".

Settlement Occurs During Construction (Sketches G-2 through G-4)

Sketches G-2 and G-3 show results of calculations of the major and minor principal stresses computed from the assumptions: (1) that the entire foundation settlement occurs immediately on application of load during construction as the embankment is being built; and (2) that the embankment material is capable of sustaining tensile stress. Case A is computed on the assumption that the foundation is so rigid that it can be considered incompressible. Cases B and C are calculations for two different foundation compressibilities, resulting in maximum foundation settlements of 0.34 and 1.42 feet, respectively. Sketch G-4 shows calculated settlements of all points within the dam and foundation, for cases A, B and C.

Settlement After Construction Assuming Tension Develops in the Dam (Sketch G-5)

Sketch G-5 shows the results of calculations of the minor principal stress at every point within the embankment, computed assuming that the foundation settled a maximum of 0.34 feet during construction and then that the foundation settlement increased by various amounts after construction.

---

1) The writer is greatly indebted to Drs. Duncan and LeFebvre who made all the calculations described herein (Appendix G) as a part of their more extensive studies of this new method of calculation and supplied the results freely in a true spirit of scientific inquiry and cooperation. Complete and detailed descriptions of the assumptions and computational techniques employed are given in Kulhawy and Duncan, 1970, and are also summarized in a paper by Messrs. Kulhawy, Duncan and Seed which has been submitted for publication in the Journal of the Soil Mechanics and Foundation Engineering Division, ASCE.)

At the end of construction, stress distribution is shown in Case B, which is the same condition as Case B in Sketch G-3. Case D shows the computed distribution of minor principal stresses existing after the foundation settlement has increased from 0.34 to 0.61 feet (that is, a post-construction settlement of 0.61 minus 0.34 equals 0.27 feet). Cases E through G show similar calculations for total foundation settlements of 0.88, 1.15 and 1.42 feet.

Settlement After Construction Assuming No Tension in Dam (Sketch G-6)

In Sketch G-5, Cases D-G, it is seen that tensile stresses develop at the top of the dam. Sketch G-6 shows results of the calculations for the same examples with the assumption that the embankment material can not sustain tensile stress. For example, Case L is the same as Case G, except for this different assumption regarding tension.

The calculations summarized in Sketch G-6 are made on the assumption that when the minor principal stress enters tension, failure occurs; that is, that the minimum value of the minor principal stress is 0. The shaded areas on Sketch G-6 are those in which "tensile failure" occurs under this assumption.

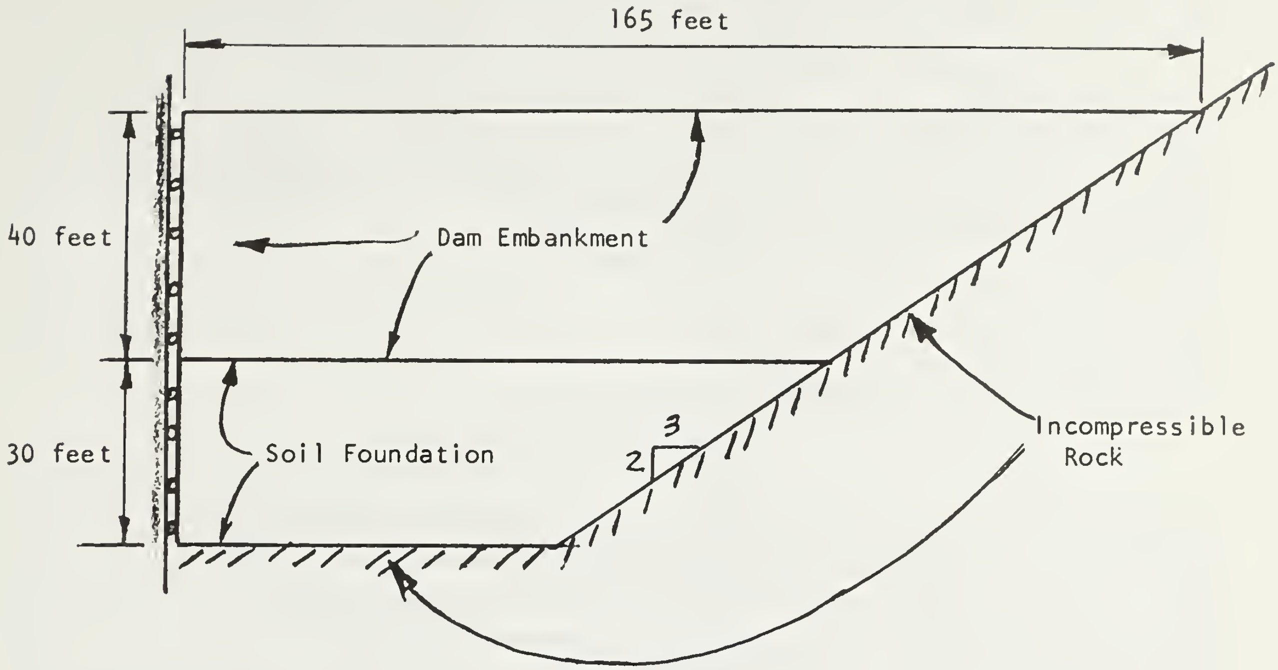
Summary of Main Results (for Dams of General Dimensions of SCS Flood Control Dams Built of Stiff Clay -- Sketch G-1)

1. Assuming the embankment can take tension, extensive zones of tensile stress develop at the end of the dam as a result of the differential foundation settlement.

2. For a given amount of foundation settlement, the size of the tensile zone and the magnitude of the tensile stresses are much greater if the foundation settlement occurs after the dam is completed than if the foundation settlement occurs during construction of the dam. (Compare Sketches G-3 and G-5.)

3. For foundation settlement of 1.42 feet (of which 0.34 feet occurs during construction) computations show a large zone of tension -- extending almost to the bottom of the embankment (Case G, Sketch G-5).

4. Computations based on the assumption that the embankment cannot take tension show a "failed" zone which is roughly the same dimensions and location as the zone of tension computed on the assumption that the embankment can take tension (Sketch G-6).

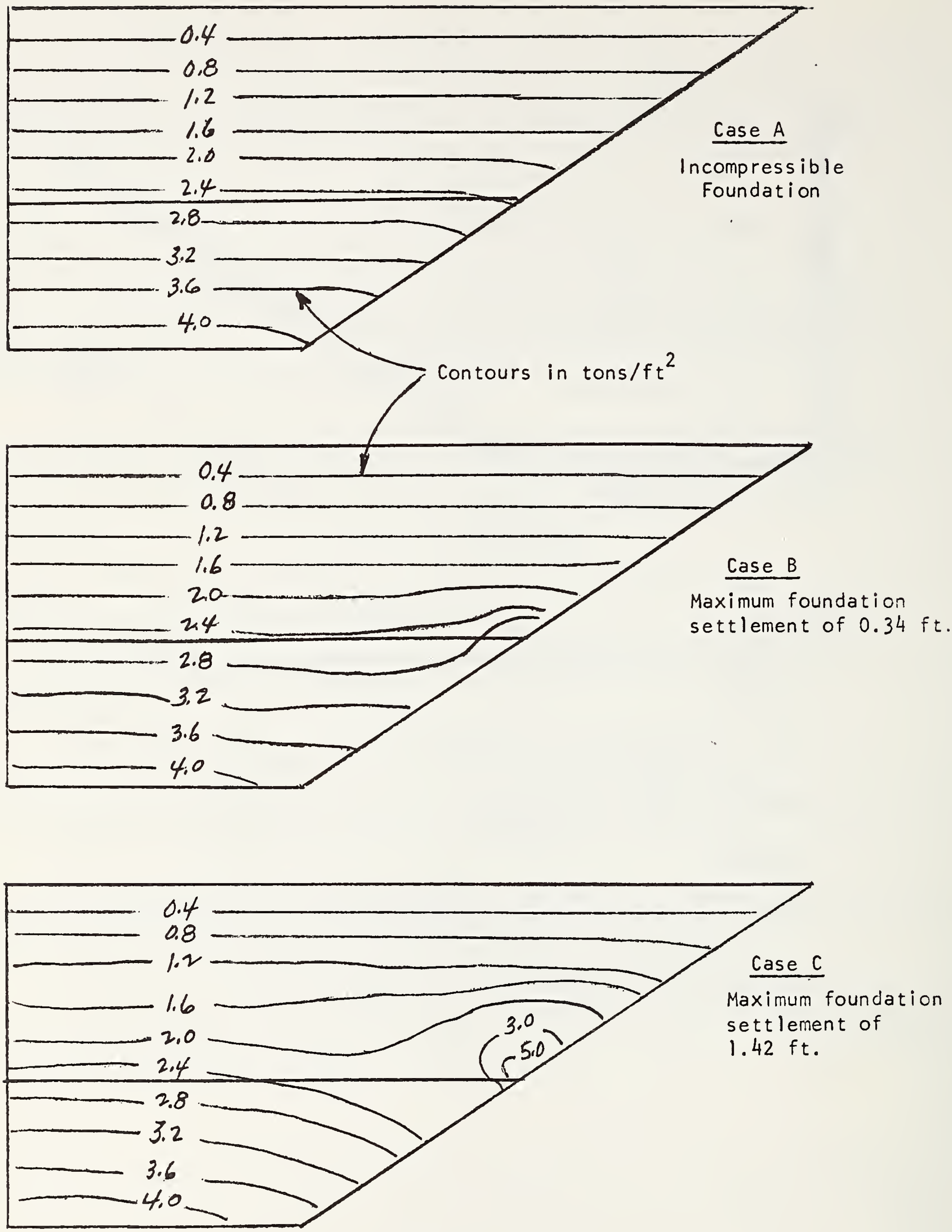


Assumptions:

1. Embankment clay has non-linear stress-strain curve which varies at every point as a function of the stress level (See Sketches G-7 and G-8).
2. Dam is considered a two-dimensional body with above shape and boundary conditions.
3. Embankment density = 112 pcf.
4. The dam is built in eight horizontal layers of equal (five foot) thickness.
5. Computations are made for two assumptions: (1) the embankment can withstand tension and (2) no tension can develop.
6. Computations are for stresses caused by weight of dam only: no influence of the reservoir is included.

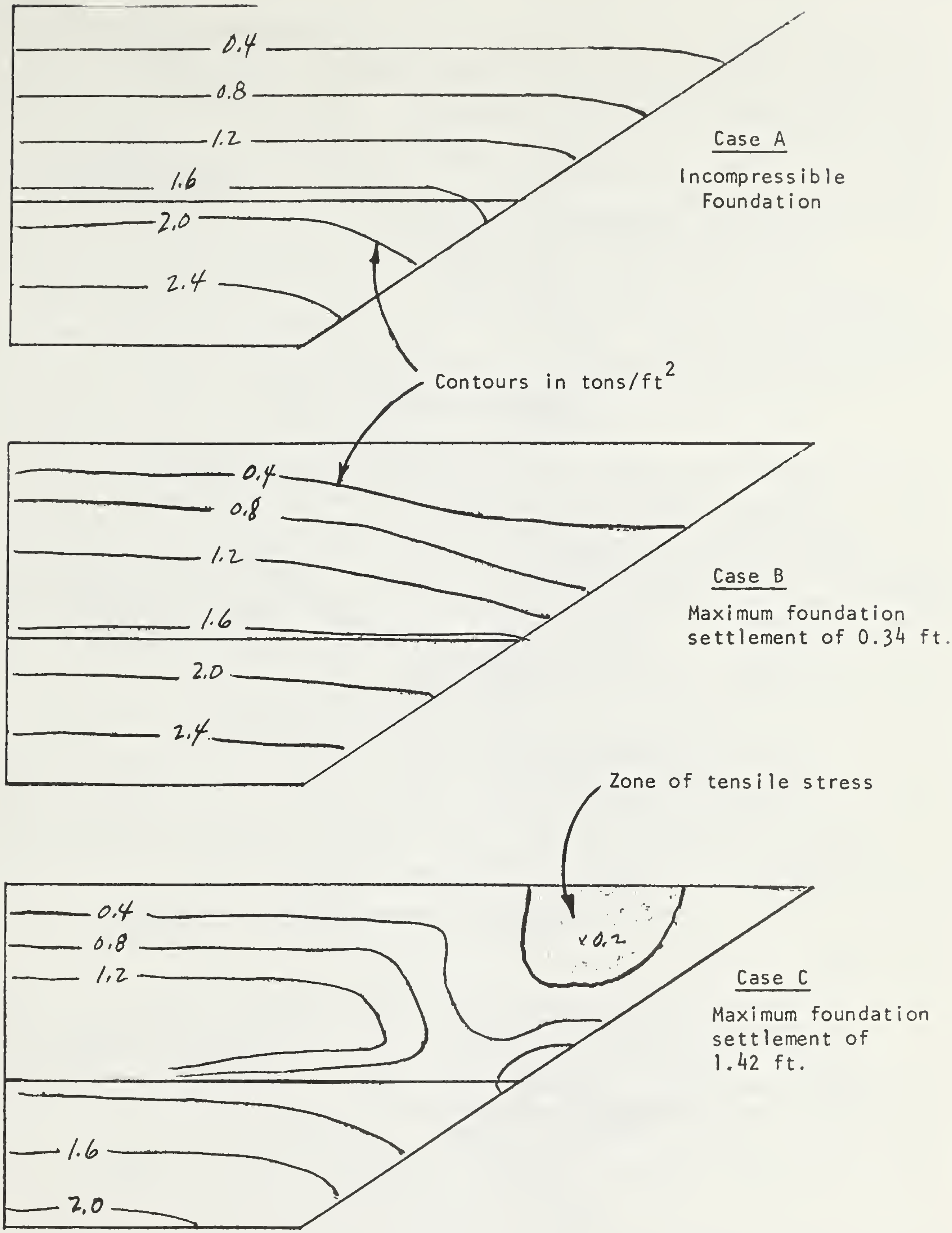
Mathematical Model and Assumptions

SKETCH G-1



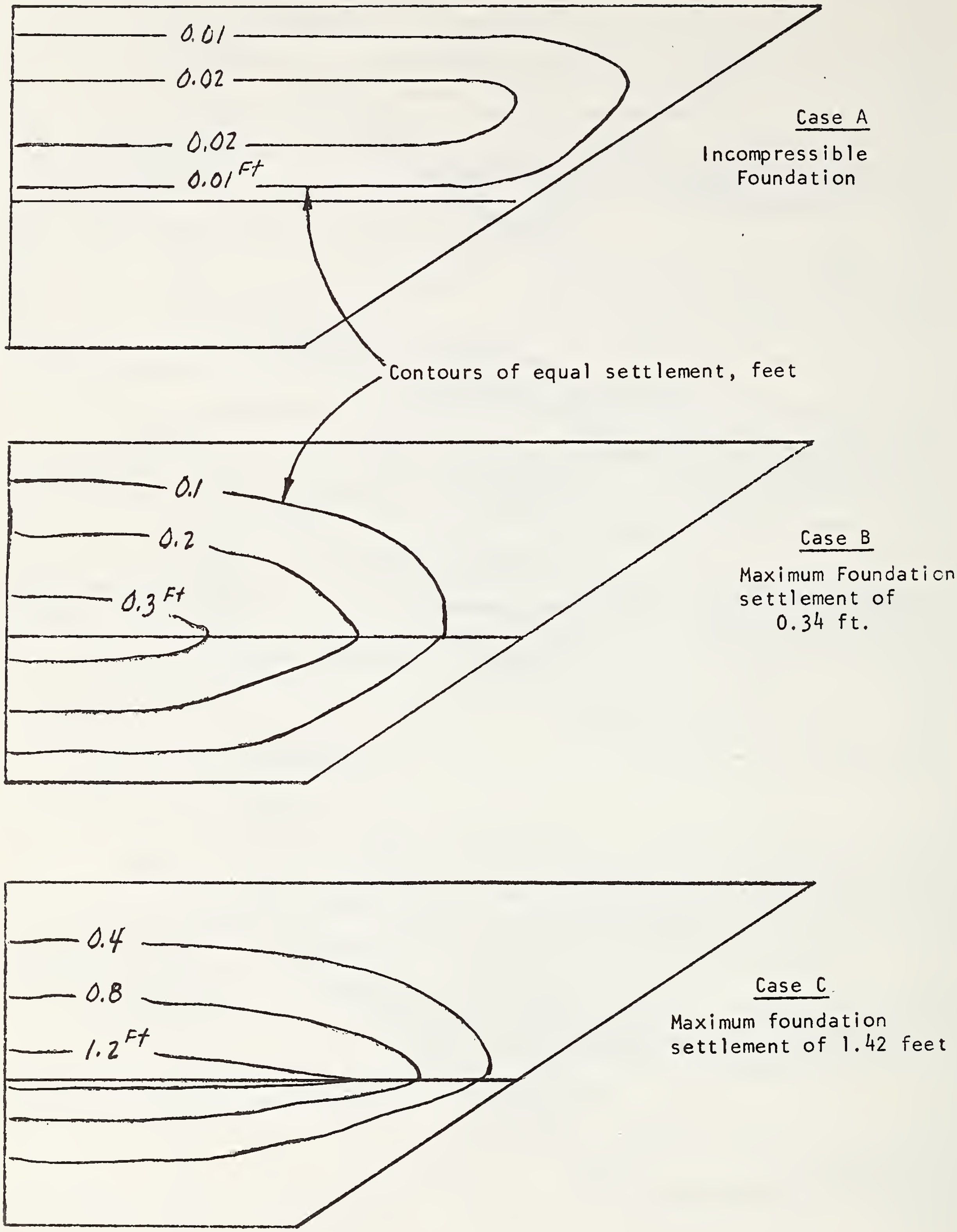
Major Principal Stresses Computed Assuming All Foundation Settlement Occurs During Construction as the Dam Is Being Built

SKETCH G-2



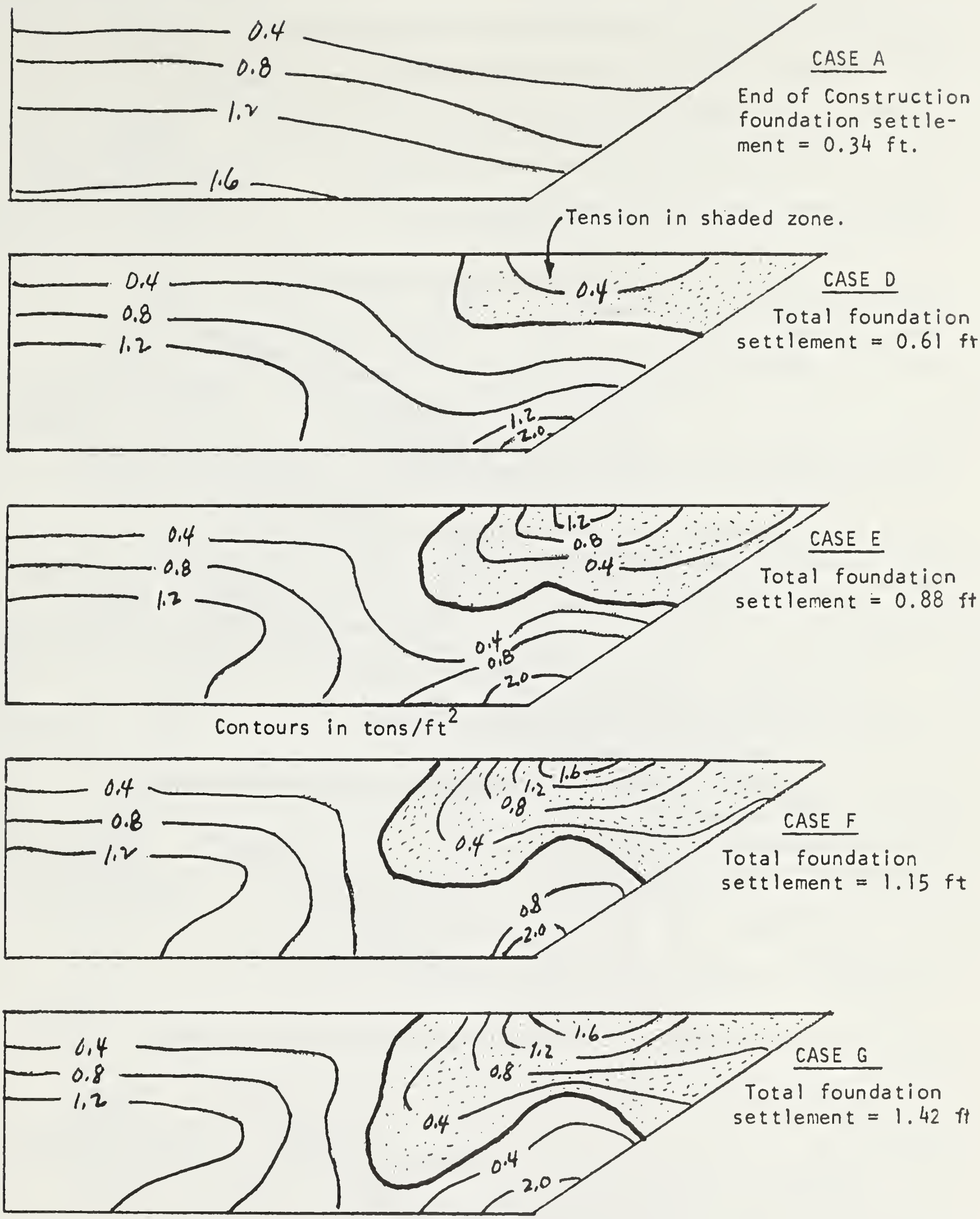
Minor Principal Stresses Computed Assuming All Foundation Settlement Occurs During Construction as the Dam Is Being Built

SKETCH G-3



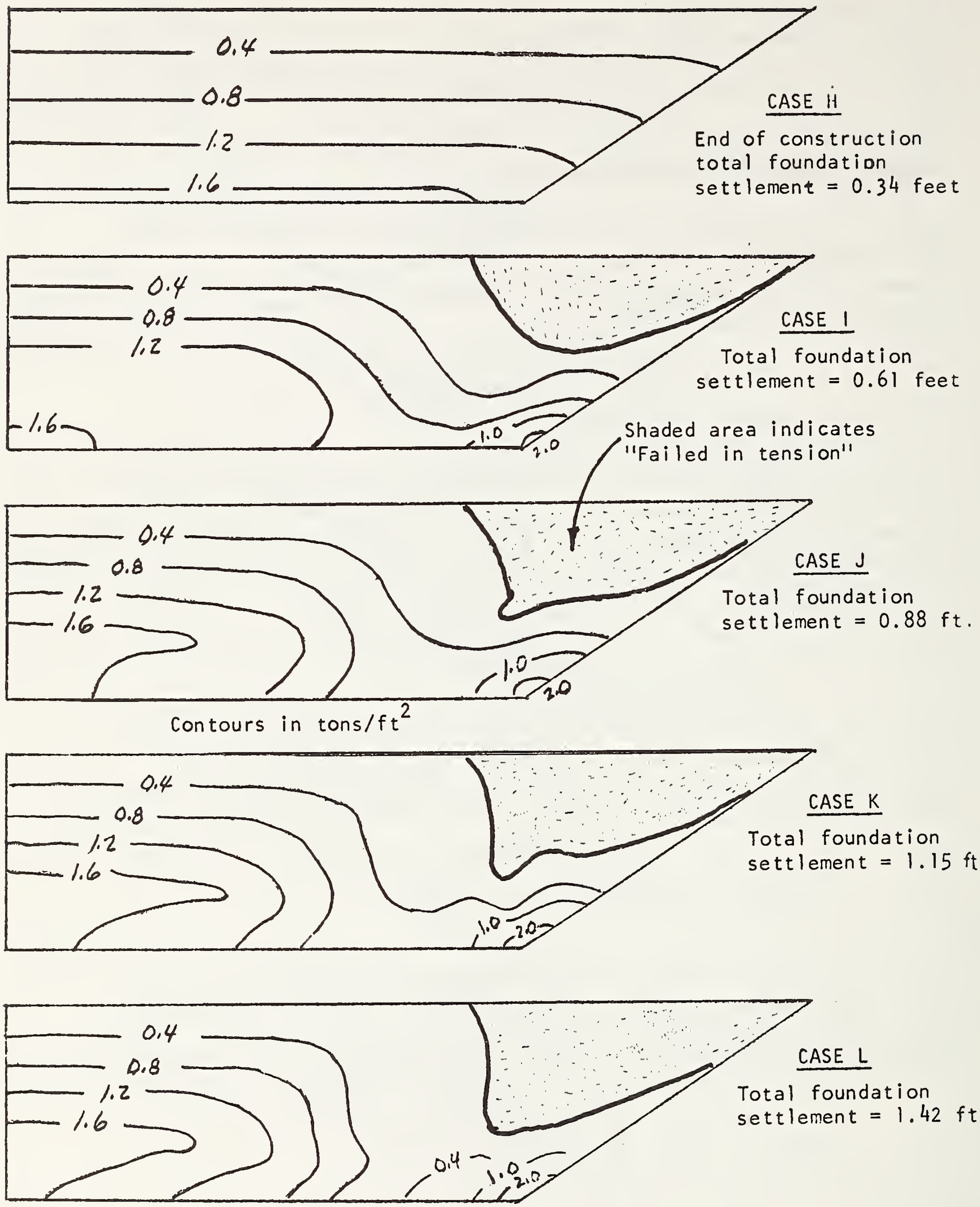
Settlement Computed Assuming All Foundation Settlement
Occurs During Construction as the Dam Is Being Built

SKETCH G-4



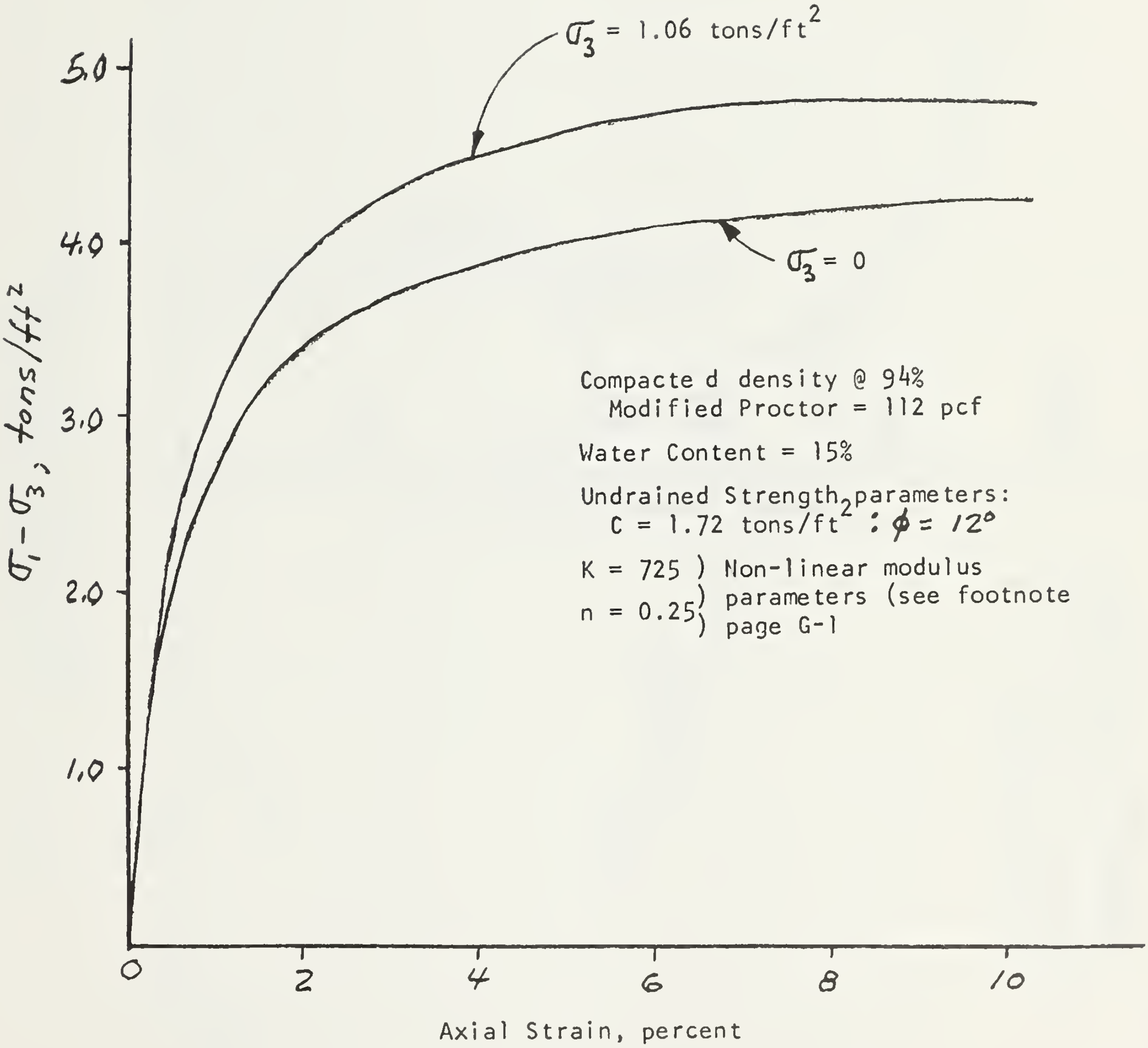
Minor Principal Stresses Computed Assuming the Foundation Settles 0.34 Feet During Construction and Continues to Settle After Construction Varying Amounts

SKETCH G-5



Minor Principal Stresses Computed in Same Way as Sketch G-5 Except for Assumption That Embankment Cannot Develop Tensile Stress

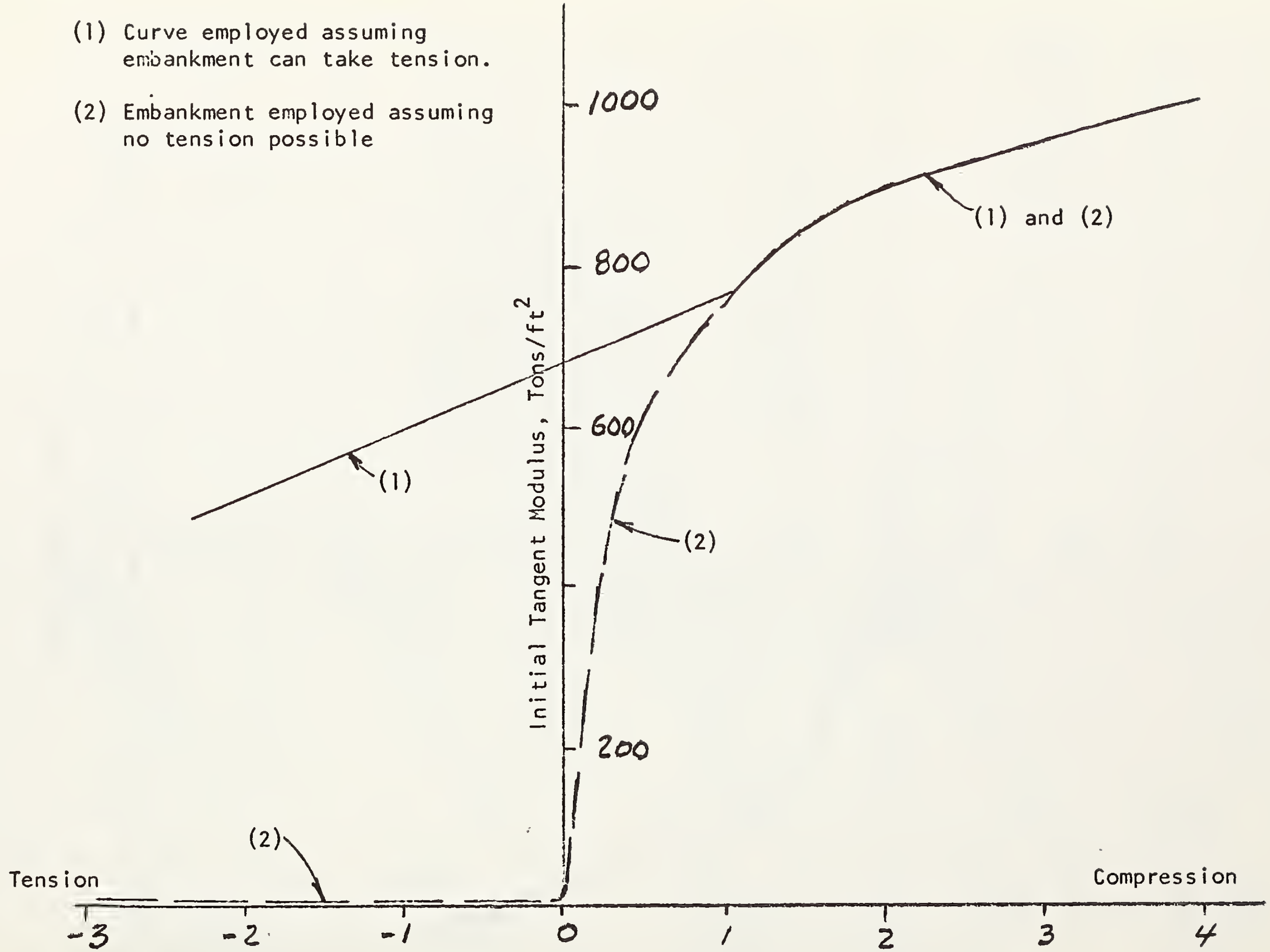
SKETCH G-6



Stress-Strain Curves for a STiff, Compacted Clay
Used as Basis for Selecting Tangent Modulii (Sketch G-8)

SKETCH G-7

SKETCH G-8



Minor Principal Stress, tons/ft^2

Variation of Initial Tangent Modulus with Minor Principal Stress Assumed in Finite Element Analysis

APPENDIX H

SOIL CONSERVATION SERVICE
LABORATORY DISPERSION TEST PROCEDURE

PROCEDURE FOR LABORATORY DISPERSION TEST USING MOIST SOIL

A. SCOPE

The purpose of this test is to provide an index of the stability of soil aggregates in water. It is designed to measure the amount of material finer than 0.005 mm. in a soil-water suspension that has been subjected to a minimum of mechanical agitation.

B. APPARATUS AND EQUIPMENT

1. U. S. Standard #10 sieve.
2. Air-tight containers for storing moist sample.
3. Balance sensitive to 0.01 grams, minimum capacity of 1000 grams.
4. 500 ml. filtering flask with side tube and one-hole stopper.
5. Vacuum pump.
6. Hydrometer jar calibrated at 1000 ml.
7. Calibrated hydrometer with a specific gravity range of 0.995 to 1.030 and marked in 0.001 subdivisions.
8. Demineralized water.
9. Thermometer accurate to 0.5° C.
10. Timer or clock.
11. Worksheet (Form SCS-360).

C. SAMPLE PREPARATION

1. The sample is collected and shipped to the laboratory in a jar, plastic-lined bag or other air-tighy container to prevent moisture loss.

2. The following steps will be performed expeditiously to hold moisture loss to an absolute minimum:

   a. If a sample is larger than two pounds, split out a representative sub-sample of approximately 500 grams.

   b. Work material through the #10 sieve. The method used will vary depending on the moisture content and type of material. Moist materials of relative low plasticity may be processed by hand rubbing through the sieve whereas a wet plastic material may have to be forced through the sieve with a putty knife. Material > #10 sieve will be discarded. Material < #10 will be placed in an air-tight container.

   c. Determine moisture content of the material < #10 by oven drying using a representative sub-sample of not less than 100 grams. Record % w.

D. PROCEDURE

1. Place approximately 125 ml. of demineralized water in the filtering flask.

2. Weigh the equivalent of 25.00 grams of oven dry soil from the container of moist material and place into the flask with water. [(25) (1.0 + w expressed as a decimal)].

(Procedure for Laboratory Dispersion Test Using Moist Soil - Cont.)

EXAMPLE: w = 16.4%
25 x 1.164 = 29.10 grams of moist material

3. Place the rubber stopper into the mouth of the flask and connect to vacuum pump.

4. Start pump and apply a full suction.

5. At approximately 3 minutes, 5 minutes, and 8 minutes after pumping has started, swirl the flask several times in a rotating manner to assist in removing entrapped air.

6. After a total pumping time of 10 minutes, disconnect from vacuum.

7. Wash the soil-water suspension into the hydrometer jar and add demineralized water to the 1000 ml. mark.

8. Shake the cylinder end over end at the rate of approximately 30 times per minute for one minute and record the time at the end of the shaking. This is the start of the sedimentation period. The time interval between item 2 and item 8 should not exceed one hour.

9. Determine the percent of material finer than 0.005 mm. in suspension.

10. Compute the percent dispersion as follows:

$$\text{Percent Dispersion} = \frac{\text{\% finer than 0.005 mm. in the soil-water suspension}}{\text{\% finer than 0.005 mm. from hydrometer analysis}} \times 100$$

11. Report as percent dispersion.

APPENDIX I

SOIL CONSERVATION SERVICE
DISPERSION TEST PROCEDURE

April, 1968

Tentative
Field Dispersion Test
(Dilution-turbidity method)

A. Scope: The purpose of this test is to determine the ratio of the fraction < 0.005 mm. in an untreated soil-water suspension to the fraction < 0.005 mm. in a soil-water suspension that has been subjected to chemical dispersion. Both suspensions receive an equal amount of mechanical agitation.

B. General: The test is made by comparing the turbidity of a fractional part (hereafter called aliquot) of a chemically dispersed soil-water suspension with the turbidity of an aliquot of an untreated soil-water suspension.

The comparison is made by diluting the chemically dispersed aliquot until the turbidity is comparable to the turbidity of the untreated soil-water aliquot.

The aliquots are withdrawn by pipet from a sedimentation cylinder at a time when only the material finer than 0.005 mm. remains in the suspension at the sampling depth.

The amount of soil finer than 0.005 mm. in the untreated soil-water suspension relative to the total amount of soil finer than 0.005 mm. in the chemically treated soil-water suspension is reported in terms of amount of dilution of chemically treated sample required to match the turbidity of the untreated suspension. This dilution ratio reflects the natural dispersibility of the clay-sized fraction.

C. Apparatus:

1. Balance sensitive to 0.01 gram with minimum capacity of 100 grams.
2. Weighing dishes. (Disposable aluminum foil dishes weighing approximately 1-1/2 grams have been used satisfactorily.)
3. Hydrometer jars - capacity 1000 ml. (Two are required for one test.)
4. Thermometers. Must include range from 18° C. to 30° C. with 1° subdivision. (Two required.)
5. Pipets: Three 25 ml, one 100 ml.
6. Two-foot length of rubber tubing for use on pipet.
7. Graduated cylinders: One 10 ml, four 100 ml.
8. Volumetric flask, 1000 ml w/stopper.

- - - - - - - - - - - - - - - - - - - -

Contribution from SCS-Soil Mechanics Unit. Prepared by K. W. Dermann, Supvry Civil Engrg Techncn, under direction of L. P. Dunnigan and R. S. Decker. Field work in checking procedure by K. W. Dermann and D. E. Baltzer, Civil Engrg Techncn.

9. Several 2 qt. jars.

10. Demineralized water - minimum of one gallon per sample tested.

11. Dispersing agent (see paragraph D).

12. Clock or watch with sweep second hand.

D. Dispersing Agent:

1. A solution of Calgon (available in most grocery stores) shall be prepared in distilled or demineralized water at the rate of 40 grams Calgon per liter.

2. Preparation of stock solutions. The dispersing agent must be prepared by thorough mixing or shaking. A convenient procedure for mixing is as follows:

    a. Add required amount of selected dispersing agent to approximately 600 ml. distilled or demineralized water in volumetric flask.

    b. Shake thoroughly (mechanically or by hand) for approximately 10 minutes or until all particles are in solution.

    c. Bring to 1000 ml. volume and shake briefly to insure uniformity throughout the solution.

    d. Shake thoroughly each time prior to use.

    e. Discard solutions that remain after 30 days.

3. Each sample to be dispersed will contain 125 ml. of the Calgon stock solution.

E. Sample Preparation:

1. Obtain minimum of 100 grams of material at natural moisture content.

2. Remove all material > #10 size and discard.

3. Shred material < #10 and retain at existing moisture content.

F. Procedure:

1. Add approximately 600 ml. of demineralized water to two 1000 ml. hydrometer jars. Label jars No. 1 and No. 2.

2. To jar No. 1 add 125 ml. of the Calgon stock solution. (Be sure to shake stock solution thoroughly immediately prior to use.)

3. Weigh two 28.0 gram sub-samples of the moist material prepared in 1 above. Add one of the sub-samples to each of the hydrometer jars.

4. Bring both jars to 1000 ml. volume with demineralized water.

5. Shake each jar end-over-end for one minute to wet and disperse the material uniformly.

6. Allow samples to soak for 15 minutes.

7. Shake jar No. 1 end-over-end for one minute to insure thorough mixing of the suspension. The number of turns during this minute should be approximately 60, counting the turn upside down and back as two turns. Any soil remaining in the bottom of the cylinder during the first few turns should be loosened by vigorous shaking of the cylinder while it is in the inverted position. At the end of the one-minute shaking period, set the cylinder on a stable, convenient work table. Record exact time cylinder is set on table as this is the time that sedimentation starts. Repeat this procedure for jar No. 2, allowing an interval of 5 minutes to elapse. This will provide adequate time to rinse pipet, etc.

8. Insert thermometers to determine temperature during sedimentation period.

9. Using 25 ml. pipet with rubber tube, withdraw a 25 ml. aliquot from each of the soil-water suspensions at a 5 cm. or 10 cm. depth at the appropriate time based upon the observed temperature during the sedimentation period. (See attached Table 1.) The sedimentation time is the same for dispersed and non-dispersed suspensions. The aliquot must be withdrawn carefully from the selected depth. Approximately 30 seconds is recommended for withdrawal of a 25 ml. aliquot.

10. Drain each aliquot into separate 100 ml. graduated cylinders.

11. Rinse pipet with demineralized water after each use and discard waste.

12. Visually compare turbidity of the two suspensions to determine portion of chemically dispersed aliquot to use for dilution. If there appears to be little difference in turbidity, use a 10-25 ml. representative portion of the dispersed aliquot. If there appears to be a great contrast, use 2-5 ml. of the dispersed solution. Measure out smaller quantities of dispersed suspension in a 10 ml. graduate and pour into 100 ml. graduated cylinder. (All transfers of suspension should be preceded by swirling and agitation of the suspension to insure that the smaller quantities transferred contain a representative amount of soil particles.) <u>Record initial quantity of dispersed suspension selected for dilution.</u> (V_1)

13. Gradually and carefully add demineralized water to the suspension selected in 12 above to bring to the same turbidity (by visual observation) as the 25 ml. aliquot of the non-dispersed solution. (The best background for visual comparison of turbidity is natural light.) The dispersing agent has a tendency to cause a slight color change in some soils. Therefore, a visual comparison of the turbidity is necessary.

14. Record total quantity of soil-water suspension (dispersed sample) when dilution is completed. (V_2)

G. Reporting and Interpreting:

Calculate the dilution ratio as follows:

$$\text{Dilution Ratio} = \frac{V_2 = \text{Total quantity of diluted dispersed suspension (from 14 above)}}{V_1 = \text{Initial quantity of undiluted dispersed suspension (from 12 above)}}$$

A low dilution ratio indicates that the clay-sized fraction (< 0.005 mm.) of the soil is easily dispersed without chemical additives. A high ratio indicates low dispersibility.

Tentative correlations from Figure 1 indicates the following criteria:

Soil Classification (Unified System)	Critical Dispersibility % Dispersion (Laboratory)	Critical Dispersibility Dilution Ratio (Field)
ML, SM, SC	20	< 13
CL	30	< 8
CH, MH	40	< 5

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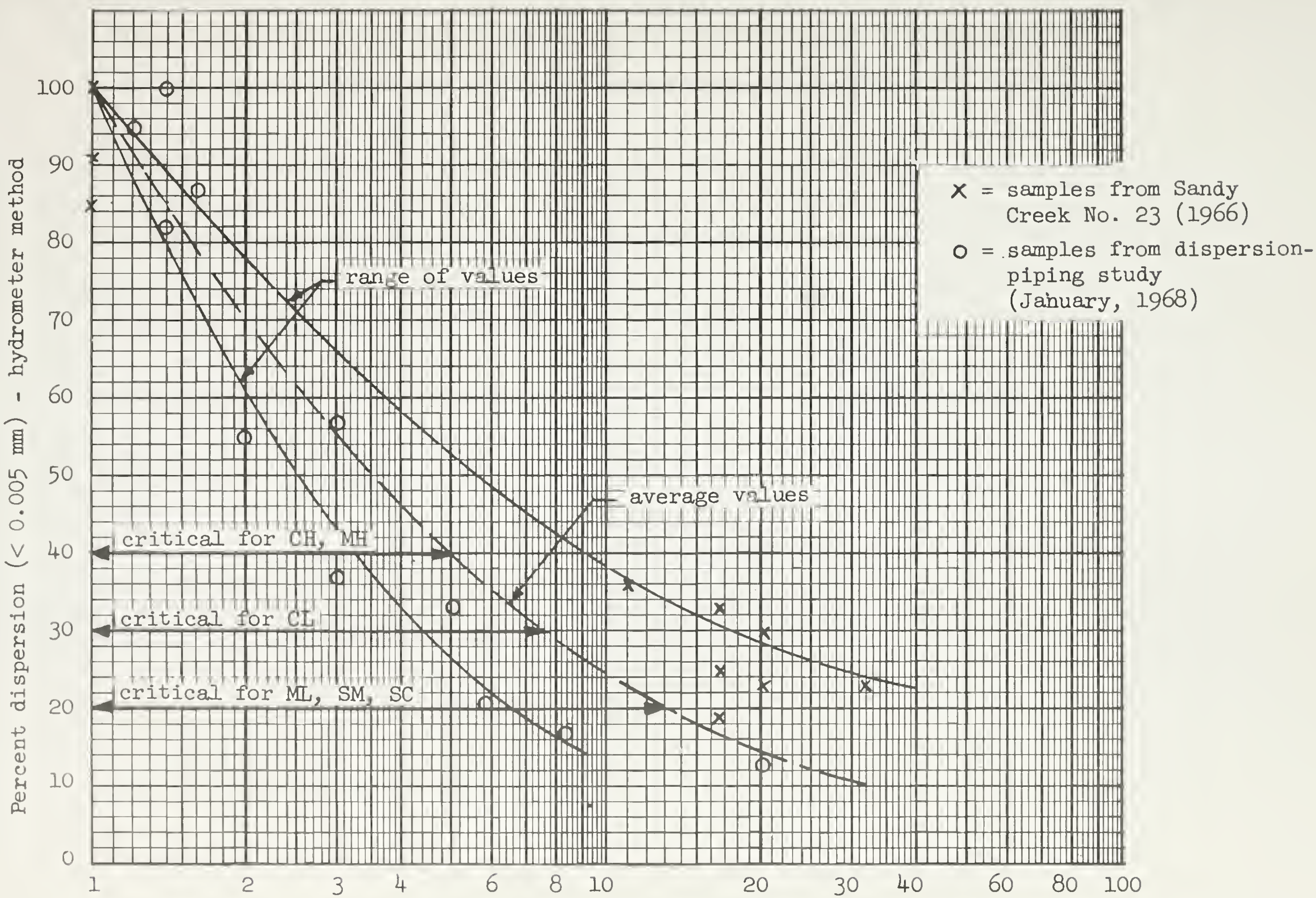
TABLE 1

Sedimentation Table - Pipet

Sampling Times at 10 cm. and 5 cm. Depths for Particle Diameter of 0.005 mm.

Temp °C	$\left(\frac{.005}{k}\right)^2$ C	Sampling Time 10 cm. Min.& Sec.	Sampling Time 5 cm. Min.& Sec.
18	0.1277	78-0	39-0
19	0.1308	76-30	38-0
20	0.1342	74-30	37-0
21	0.1376	72-30	36-30
22	0.1409	71-0	35-30
23	0.1441	69-30	35-0
24	0.1477	68-0	34-0
25	0.1512	66-0	33-0
26	0.1544	65-0	32-30
27	0.1579	63-30	31-30
28	0.1615	62-0	31-0
29	0.1652	60-30	30-0
30	0.1688	59-0	29-30



Dilution ratio = $\frac{V_2}{V_1}$ = $\frac{\text{final volume dispersion suspension}}{\text{initial volume dispersion suspension}}$

Figure 1. Relationship of percent dispersion by hydrometer method and dilution ratio by field dispersion test (tentative)

APPENDIX J

PHOTOGRAPHS

Photo 1: General view of dike from helicopter.

Photo 2: Typical view of vertical tunnel from rainfall erosion in dike crest in badly damaged zone.

VENEZUELA FLOOD CONTROL DIKE

Sherard photos.

Photo 3: General view of crest showing erosion tunnels.

Photo 4: Group of vertical erosion tunnels on crest, averaging about 9 inches diameter.

BIG SAND CREEK SITE 8 DAM (MISSISSIPPI)

David D. Lynch photos.

Photo 5: Big Sand Creek Site 8 Dam. Showing typical erosion tunnel on upper part of downstream slope. About 24 in. diameter. Note good grass protection on slope.

Photo 6: Grenada Dam, Miss. Showing typical erosion tunnel on downstream slope. About 24 in. diameter. Protective grass cover is mowed several times per year and is in excellent condition.

Sherard photos.

Photo 7

Erosion tunnel, 12 in. diameter, on crest of Leader Middle Boggy Creek Site 29 in spite of excellent grass protection.

Photo 8: Wister Dam, Oklahoma. Typical, 18 in. diameter erosion tunnel on downstream slope. Excellent grass cover doesn't prevent tunnel erosion.

Sherard photos.

Photo 9: Typical views of erosion in excavated slope of natural soil in area of highly dispersive soils in vicinity of breached Oklahoma Dams.

Photo 10: Note similarity to Mississippi erosion, Photos 11 and 12.

Sherard photos.

Photo 11

Typical view of erosion pattern in excavated slope in area of highly dispersive clay in Mississippi. Note alluvial "fan" of sand at bottom.

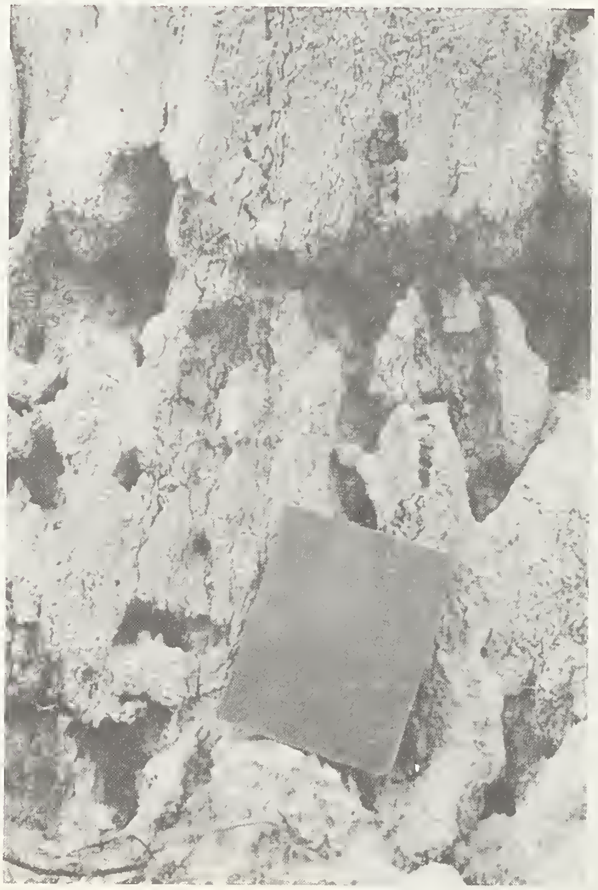


Photo 12: Closeup view of eroded face of Photo 11. Note internal tunnelling in clay deposit resembles limestone solution caverns.

Sherard photos.

Photo 13

Looking downstream at breach (see also Sketch F-11)

Photo 14

Looking upstream at breach

UPPER CLEAR BOGGY CREEK SITE 50 DAM
Oklahoma

Sherard photos.

Photo 15

Left wall of failure breach. Note 24-inch diameter erosion tunnel.

Photo 16

Erosion tunnel in left wall of breach shown above.

UPPER CLEAR BOGGY CREEK SITE 50
Oklahoma

Sherard photos.

Photo 17

Looking downstream at the entrances of two piping tunnels in the upstream slope

Photo 18

Looking upstream at tunnel exits.

Views Showing failure of Lower Middle Clear Boggy Creek Site 33 Dam, Oklahoma.

N. L. Ryker photos.

Photo 19

Nov. 20, 1964
9:15 a.m.

Photo 20

Nov. 20, 1964
10:15 a.m.

Caney Coon Creek Site 2 Dam, Oklahoma. Main erosion tunnel at the right abutment 24 hours after the first small leak was observed.

N. L. Ryker photos.

Photo 21
Downstream slope showing piping tunnels emerging on both sides of concrete pipe spillway conduit.

Photo 22
Collapse of roof of piping tunnel causes sinkhole on construction surface.

Potacocowa Creek Site 3 Dam, Mississippi. Failed by piping along the conduit before construction was completed when reservoir raised during rainstorm to an elevation which was only 4 feet above the conduit invert at the upstream end. August 25, 1962.

H. C. Huey photos.

Photo 23

Upstream face of small dam in Australia showing simultaneous development of three independent piping tunnels in dispersive clay dam at about the same elevation in the embankment. (O. G. Ingles photo)

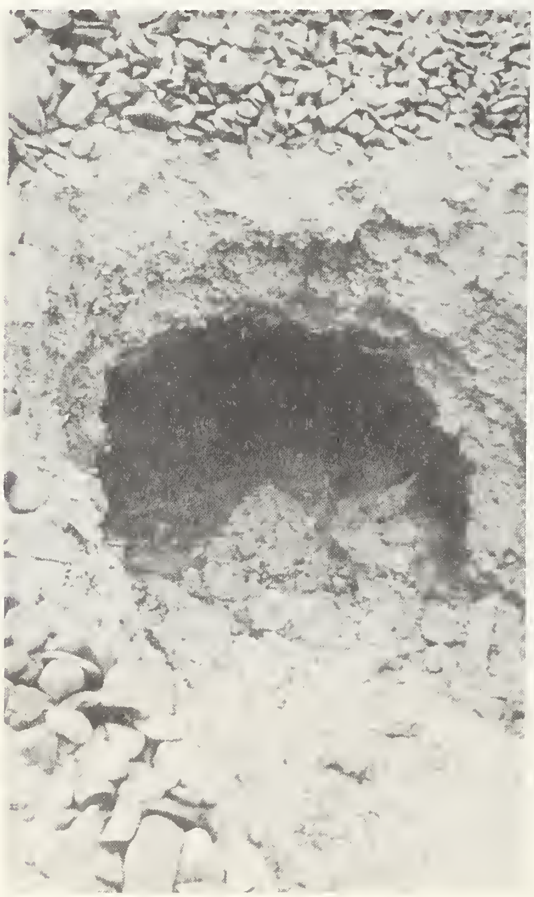


Photo 24
Entrance of piping tunnel in well constructed clay dam (Morwell Dam). The initial leak believed to have developed in a drying crack at a level where the construction work was suspended 3 months during dry weather (see Appendix A).

Photos courtesy G. D. Aitchison, CSIRO, Australia.

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